STABILITY CALCULATION FOR THE SURROUNDING ROCK OF POWERHOUSE CAVERN

[UPPER TRISHULI-1 HEP (216MW)]

2023. 02.01

DOOSAN Enerbility

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1 General

1.1 Introduction

The underground powerhouse is located within the right bank mountain of Trishuli river with vertical buried depth of about 300m. At present, overall size of powerhouse cavern is L80.0m×W20.6m×H39.4m, size of main transformer cavern, which is located at downside of powerhouse cavern and with net rock wall thickness of about L27.75m, is W15.6m×H12.3m.

According to the previous geological information and contractor's interpretation, inter bedded mica schist and quartz-schist are developed / distributed along cavern axis, as per exposed geological conditions of investigation tunnel end and main access tunnel middle, rock classes are highly assumed as Type II or even partially Type I, Type III may be developed locally.

The proposed supports of powerhouse and transformer caverns are shown in Figure 1-1~1-2. The staged excavation is shown in Figure 1-3.

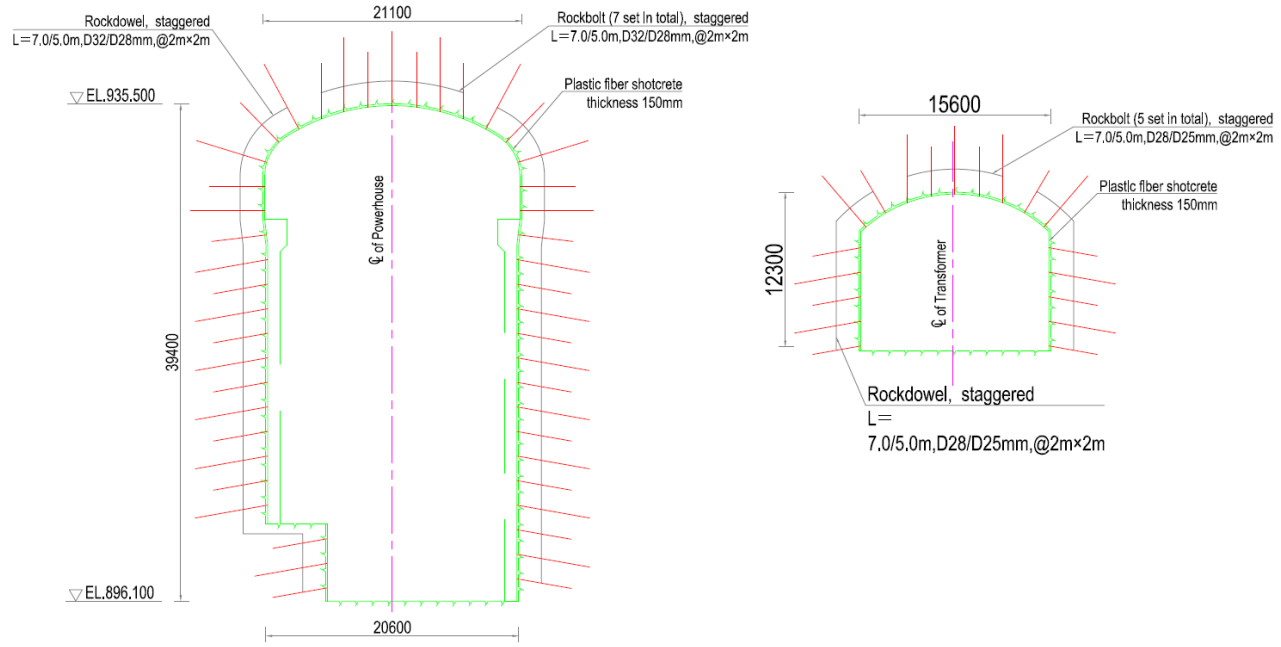


Figure 1-1 Proposed supports of powerhouse and transformer caverns (rock type II)

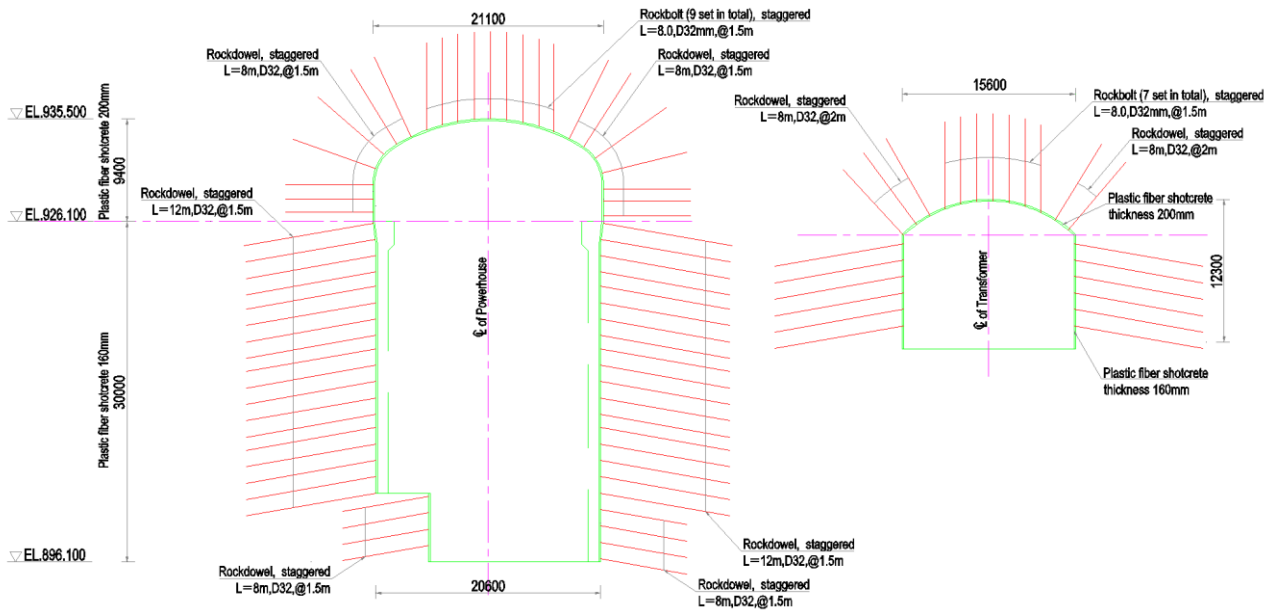


Figure 1-2 Proposed supports of powerhouse and transformer caverns (rock type III)

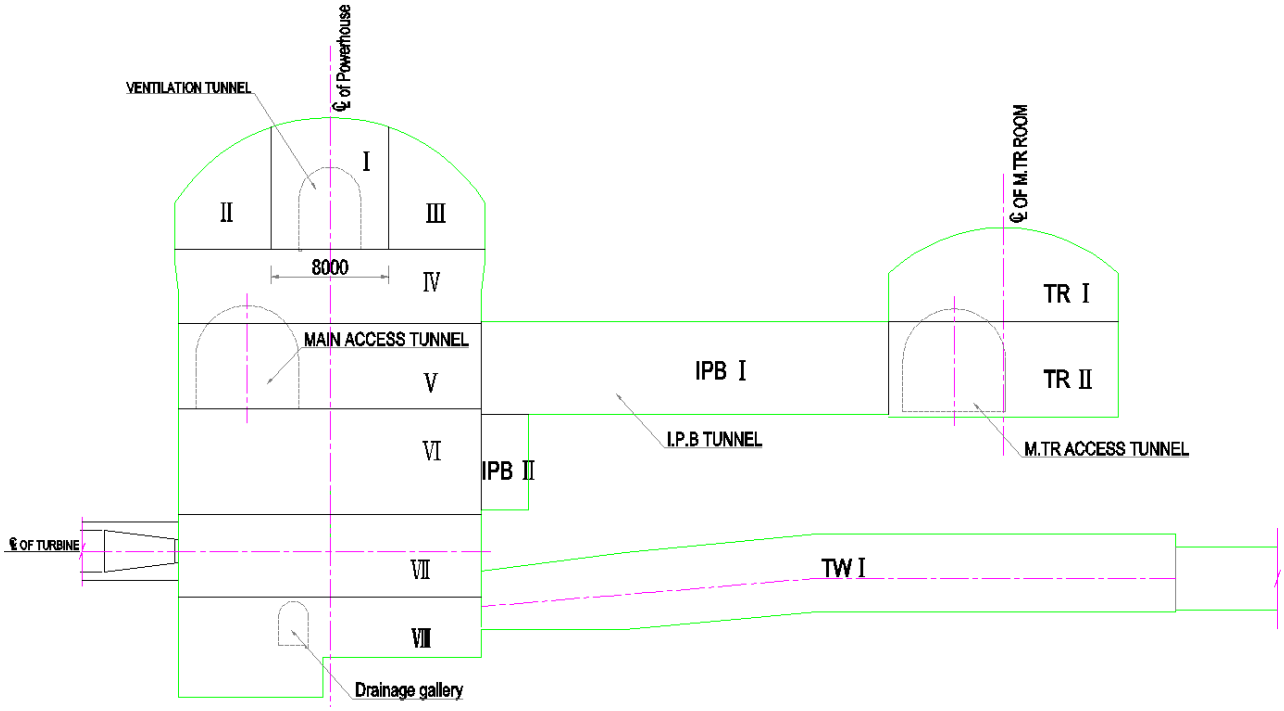


Figure 1-3 Staged excavation plan of powerhouse and transform cavern

1.2 Design Objective

This calculation is aim to verify the proposed supports include rock bolts/dowels and shotcrete be effective and feasible; ensure stability of caverns and safety of construction activities during cavern excavation.

Further 3-D stability analysis is planned to be conducted to provide more reliable information to support design of powerhouse and main transformer caverns.

1.3 References

(1) Contract documents, Employer's Requirement

(2) Tunnels and Shafts in Rock (EM 1110-2-2901)

(3) Regarding general Powerhouse size determination confirmation UT1-HEP-PC-DHI-2022-0271

(4) High Strength Deformed Steel Bars and Wires for Concrete Reinforcement Specification (Fourth Revision) (IS 1786-2008)

(5) Building Code Requirements for Structural Concrete and Commentary (ACI 318)

(6) Monitoring layout drawing of underground powerhouse and transformer cavern UT1-C-845-CVL-DG-70002

2 Theoretical Background of Calculations

2.1 Software

Software Phase 2 is applied to conduct the calculation. Phase2 is a powerful 2D finite element program for soil and rock applications. Phase2 can be used for a wide range of engineering projects including excavation design, slope stability, groundwater seepage, probabilistic analysis, consolidation, and dynamic analysis capabilities.

Software UNWEDGE is an interactive software for the stability analysis of three-dimensional wedge formed by structural discontinuity and underground excavation, which is used to analyze the underground excavation problem with discontinuous structural plane in rock mass. UNWEDGE calculates the safety factor of potentially unstable wedge, and can analyze the influence of support system on wedge stability.

2.2 Modeling Conditions, Material Law, Constitutive Model

The modeling conditions are plane strain. The material law is elasto-plastic combined with strain-softening (for rock). The rock mass is modeled as a continuum, homogeneous and isotropic. The failure criteria are the Mohr-Coulomb failure criteria. What needs to be emphasized is that such continuum model with

homogeneous and isotropic material is a model, only, and does not necessarily reflect reality. In reality, the displacements occur in blocky rock masses always along discontinuities (joints, foliation etc.), which leads to loosening of the rock mass. The determined plastic zone in the continuums model represents an area of the rock mass where rock blocks were displaced to each other (= loosening of rock mass). In reality, it does not represent an area where the material was subject to plastic flow, as in soil.

2.3 In situ horizontal stress

For unavailability of field measurement of in situ stresses at the area, gravitational stresses are adopted.

The vertical stress due to the overlying rock is:

$$\sigma_z = \gamma h$$

Where γ represents the density that is the unit weight of the rock.

The horizontal in situ stresses also depend on the depth below surface. They are generally defined in terms of the vertical stress as follows:

$$K_0 = \sigma_h / \sigma_v$$

Where K_0 represents the lateral rock stress ratio. Since there are three principal stresses directions, there will be two horizontal principal stresses. In an undisturbed rock mass, the two horizontal principal stresses may be equal.

In this calculation, K_0 is taken as 1.3 according to topographic and geological conditions of the engineering area.

3 Properties of Materials

3.1 Rock Mass

The mechanical parameters of surrounding rock are shown in Table 3-1 as per approval letter RN 0031.

Table 3-1 Mechanical parameter for surrounding rock of powerhouse caverns

Class of surrounding rock	Q	Unit weight	Poisson's ratio of rock mass	Elastic modulus of rock mass	Rock mass/rock mass shear (shearing) strength				Uniaxial compressive strength of rock mass	Uniaxial compressive strength of intact rock	Tensile strength of rock mass
					Peak		Residual				
		γ (kN/m^3)	μ	Em (GPa)	*C* (MPa)	*Φ* (°)	*Φr* (°)	*Cr* (MPa)	UCS rm (MPa)	INTACT UCS r (MPa)	σ_{tm} (MPa)
II	10~40	27.0	0.25	20.5	2.5	48	38	0.95	13.0	60	1.3
III	4~10	26.5	0.25	10.0	1.0	45	35	0.35	4.8	60	0.48

3.2 Rock Dowel/Rock Bolt

1) Dowel type: cement mortar full length bonded dowel

2) Yield strength: 500MPa

3) Tensile strength: 545MPa (Table 3 of "IS 1786-2008")

4) Elastic modulus: 210000MPa

5) Allowable tensile strength of rock bolt steel: 500MPa/1.5=333MPa. The value 1.5 is safety factor.

3.3 Shotcrete

The mechanical parameters of plastic fiber shotcrete are shown in Table 3-2. Elastic modulus is calculated by formula Ec=4700*fc'^0.5 (ACI 318, section 19.2.2 Modulus of elasticity).

Table 3-2 Mechanical parameter of plastic fiber shotcrete

Compressive strength fc'(MPa)	Tensile Strength (MPa)	Elastic modulus Ec (MPa)	Unit Weight (kN/m^3)	Poisson ratio
25	2.5	23500	24.0	0.2

4 Cavern Support

4.1 Cavern Support per Excavation Class

The proposed supports for each rock type are shown in Table 4-1.

Table 4-1 Proposed supports for each rock type

Item	Unit	proposed supports	
		II	III
Shotcrete thickness	mm	150mm	Roof :200mm Side wall: 160mm
Shotcrete strength	MPa	25	25
Powerhouse cavern supports		Roof and side walls: L=7.0m,φ32mm/L=5.0m,φ 28mm staggered. Amongst, rock bolts are proposed in the central portion of the cavern crown.	Roof: L=8.0m,φ32mm Side walls: L=12.0m,φ32mm Lower side walls: L=8.0m,φ32mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.
Transformer cavern supports		Roof and side walls: L=7.0m,φ28mm/ L=5.0m,φ 25mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.	Roof: L=8.0m,φ32mm Side walls: L=12.0m,φ32mm. Amongst, rock bolts are proposed in the central portion of the cavern crown.
Rock dowels and/or rock bolts spacing within each row	m	2.0	1.5
Rock dowels and/or rock bolts row spacing	m	2.0	1.5

4.2 Safety Factor

The support safety factor for shotcrete is 1.2.

The minimum safety factor for wedges analysis is 2.0.

5 Model Set-up, Boundary and Loading Conditions, Stress Initiation

For the case of gravitational loading, only, boundary and loading conditions to be set as shown below. The lower boundary of the counted domain is fixed in the X and Y directions, two lateral boundaries fixed in the X direction but unconstrained in the Y direction, and the top is free. The load of overburden above the top boundary is automatically applied by the software according to the elevations. Initial stress conditions in the

model can be calculated by gravitational loading, or by specifying the relative ground surface elevation.

The finite element model for rock class II and III is shown as Figure 5-1, and the corresponding gravitational loading is shown in the Table 5-1. In the model, the excavation sequence of the powerhouse cavern and transformer cavern is properly simulated.

The excavation sequence of the powerhouse cavern and transformer cavern is as follows:

Stage 1: Central portion of powerhouse cavern crown (8m× **9m**);

Stage 2: Left side drift;

Stage 3: Right side drift;

Stage 4: Bench down of the powerhouse cavern from EL. 926.50 to EL. 921.50, as well as transformer cavern from EL. 928.00 to EL. 921.58.

Stage 5: Bench down of the powerhouse cavern from EL. 921.50 to EL. 915.70, as well as transformer cavern from EL.921.58 to EL. 915.10.

Stage 6: Bench down of the powerhouse cavern from EL. 915.10 to EL. 908.50.

Stage 7: Bench down of the powerhouse cavern from EL. 908.50 to EL. 902.90.

Stage 8: Bench down of the powerhouse cavern from EL. 902.90 to EL. 896.10.

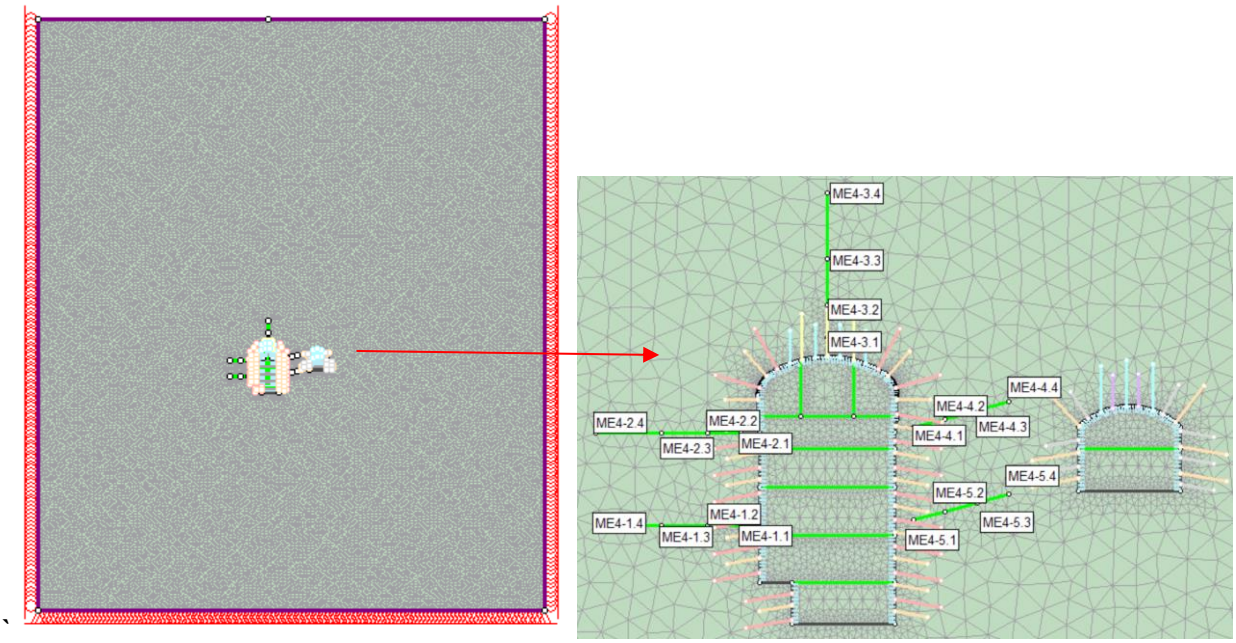


Figure 5-1 Finite element model and boundary conditions

Table 5-1 Gravitational loading for surrounding rock

Rock class	Buried depth	Rock unit weight	Overburden gravitational loading
	m	kN/m³	MPa
II	304.5	27	8.2
III	304.5	26.5	8.1

Prior to application of rock supports, stress release ratio of the surrounding rock takes 50% based on experience, which means that load to be borne by the supporting measures is assumed to be 50% of the ground in-situ stress at the corresponding location.

Initial stress filed of the simulation domain are as shown in Figures 5-2~5-3.

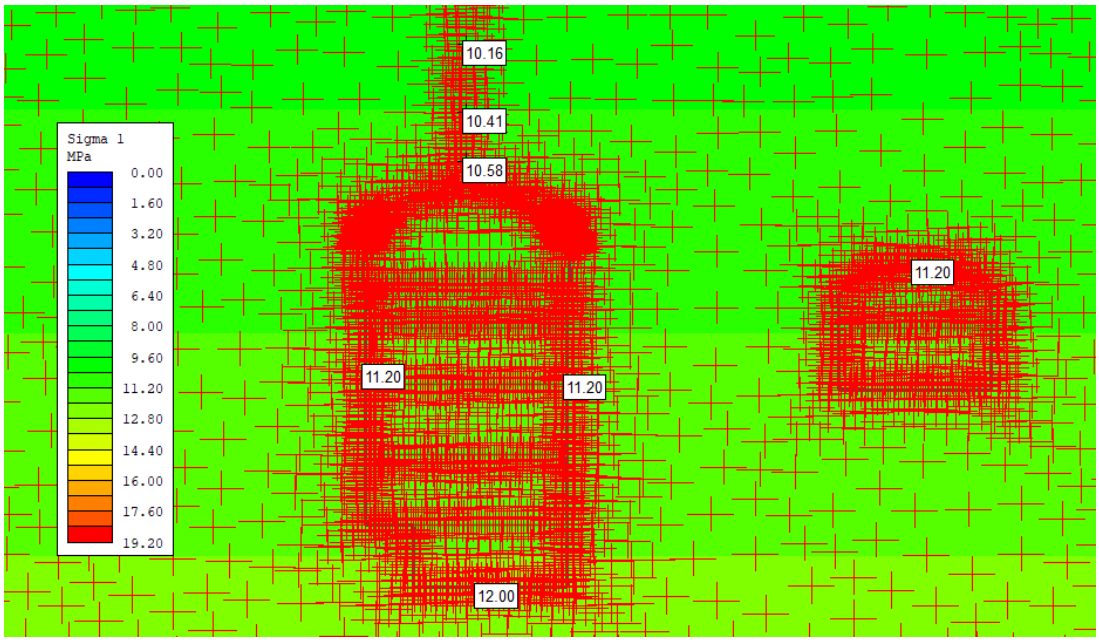


Figure 5-2 Initial stress trajectories (Rock Class II)

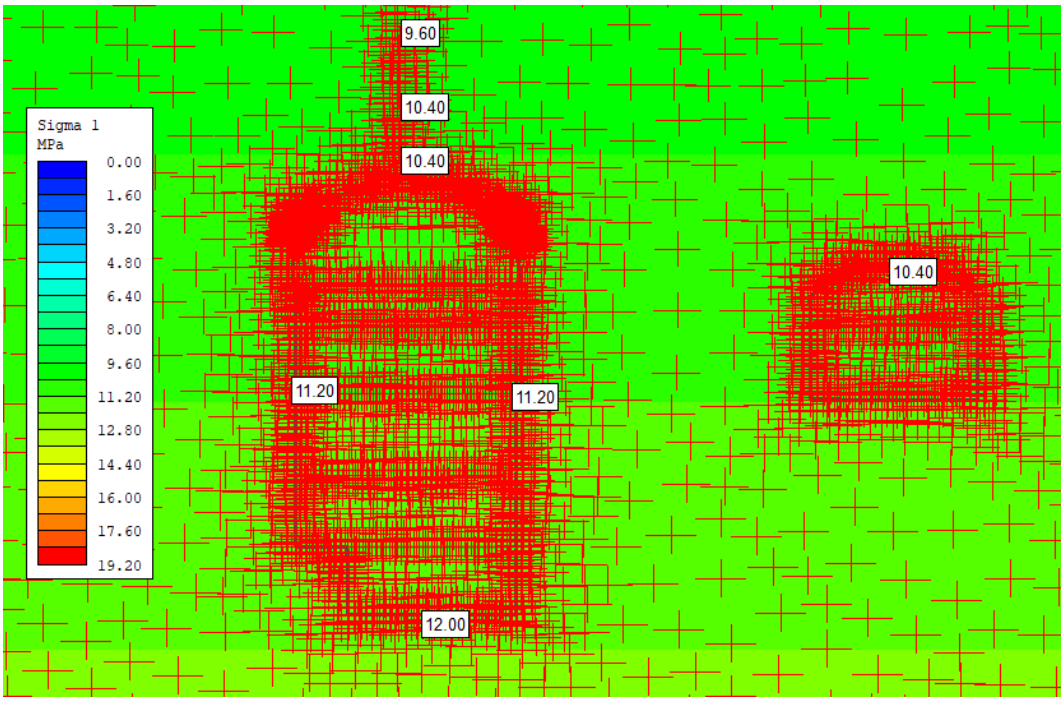


Figure 5-3 Initial stress trajectories (Rock Class III)

6 Calculation Results

6.1 Rock class II (Basic design supporting scheme)

According to bid documents, annexes, section VII, annex 5, 07 powerhouse drawings, supports of the powerhouse and transformer caverns is as below and shown in Figure 6.1-1.

(1)Powerhouse cavern

Roof: rock dowels, φ32mm, spacing 2.0m, L=8.0m.

Side wall: rock dowels, φ32mm, spacing 2.0m, L=12.0m.

(2)Transformer cavern

Roof: rock dowels, φ32mm, spacing 2.0m, L=8.0m.

Side wall: rock dowels, φ32mm, spacing 2.0m, L=12.0m

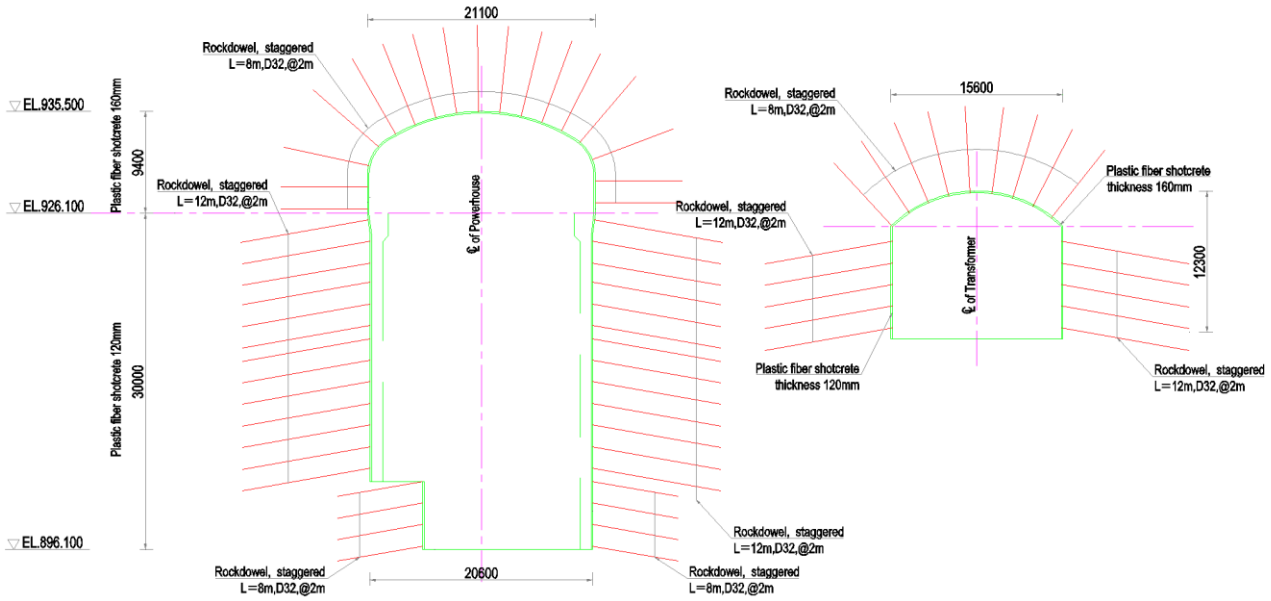


Figure 6.1-1 Basic design supports of powerhouse and transformer caverns (rock type II)

Calculation result is as shown in Figures 6.1-2~4.

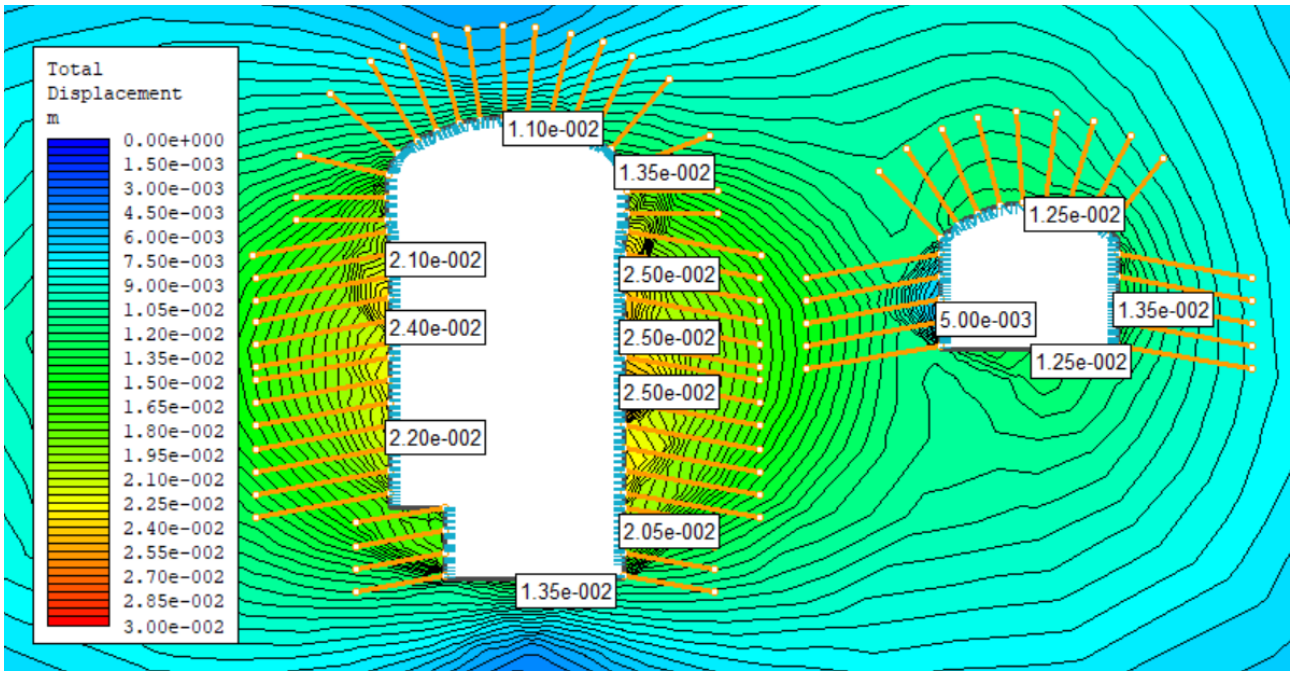


Figure 6.1-2 Total displacement after support installation (Rock Class II)

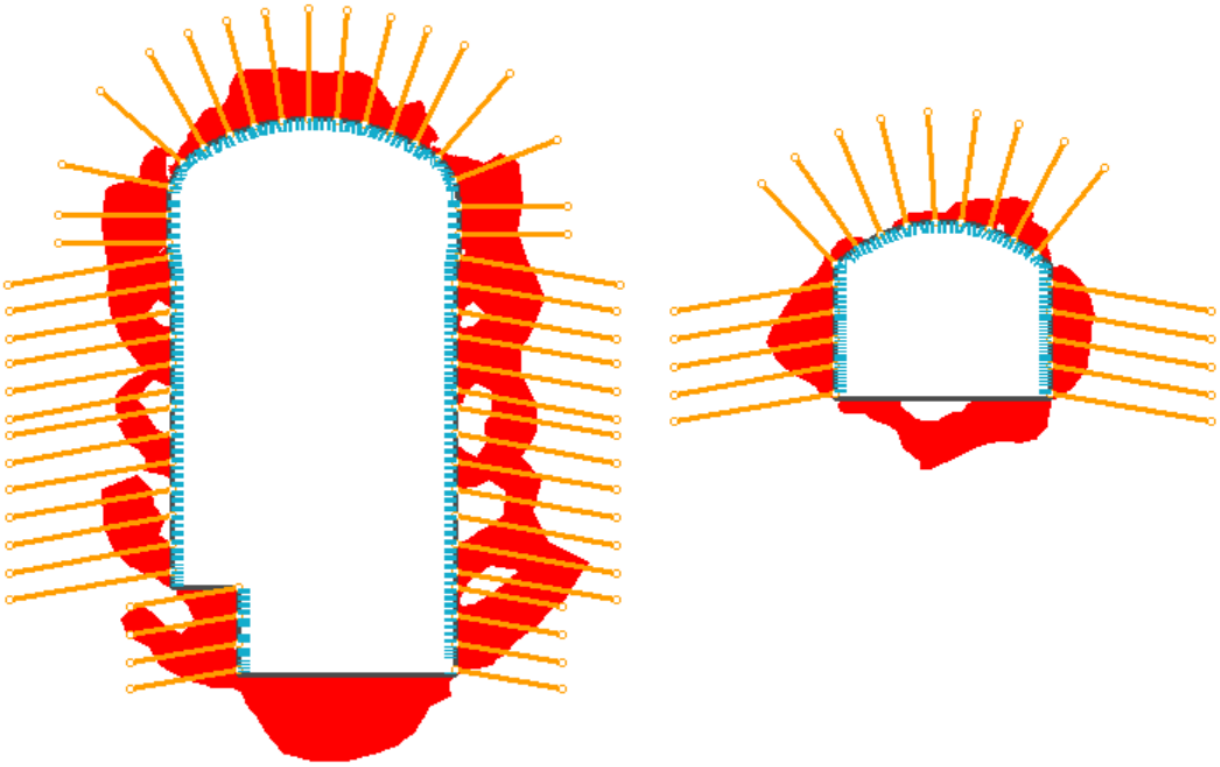


Figure 6.1-3 Plastic zone after support installation (Rock Class II)

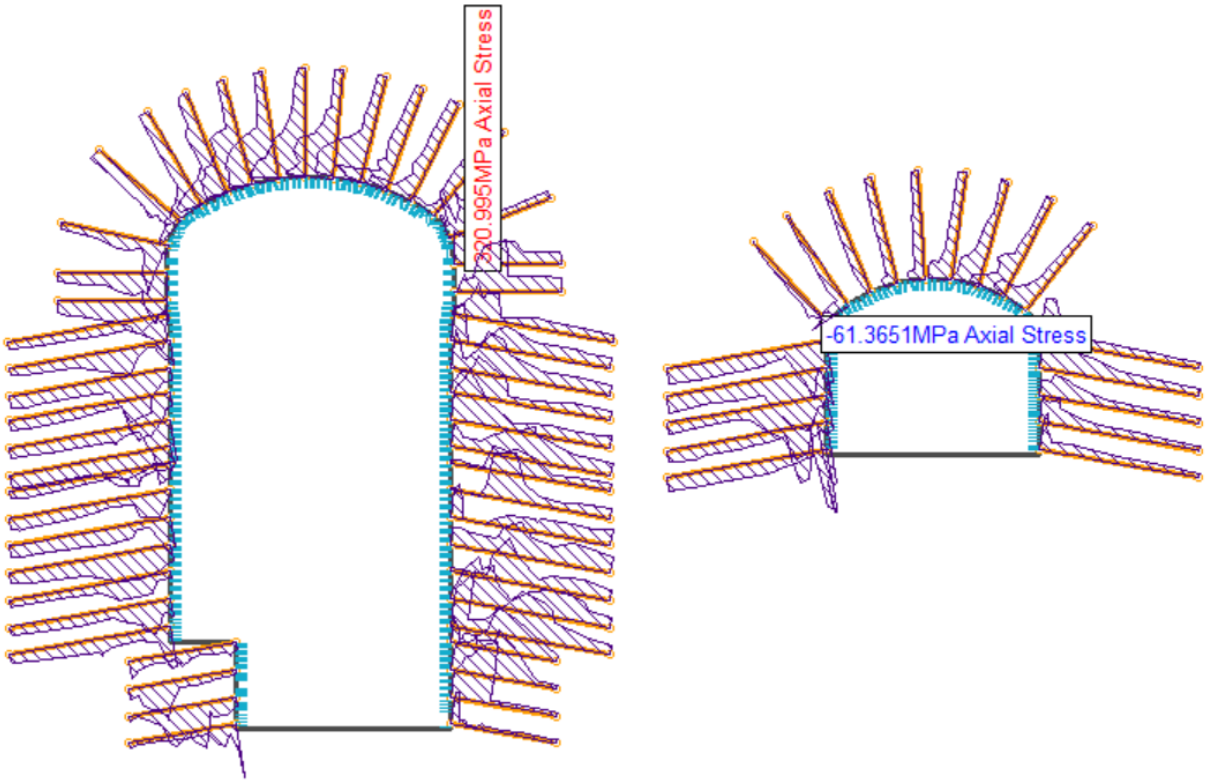


Figure 6.1-4 Axial stress of rock dowels after support installation (Rock Class II)

6.2 Rock class II (prop supports)

From the calculation results shown above, it can be seen that supports of the powerhouse and transformer caverns in class II surrounding rock were overdesigned in the basic design stage, mainly the rock dowel length, so a new set of supporting parameters is proposed as below and shown in Figure 6.2-1~6.2-3. Furthermore, in order to limit the reflection of the cavern crown effectively, rock bolts are recommended to be applied vertically at the central portion of the cavern crown instead of the rock dowels originally proposed.

(1)Powerhouse cavern

Periphery: rock dowels and bolts, φ32mm, L=7.0m/φ28mm, L=5.0m, spacing 2.0m, staggered. Pre-tensioned bolt instead of central portion of cavern crown (7 bolts).

(2) Transformer cavern

Periphery: rock dowels and bolts, φ28mm, L=7.0m/φ25mm, L=5.0m, spacing 2.0m, staggered. Pre-tensioned bolt instead of central portion of cavern crown (5 bolts)

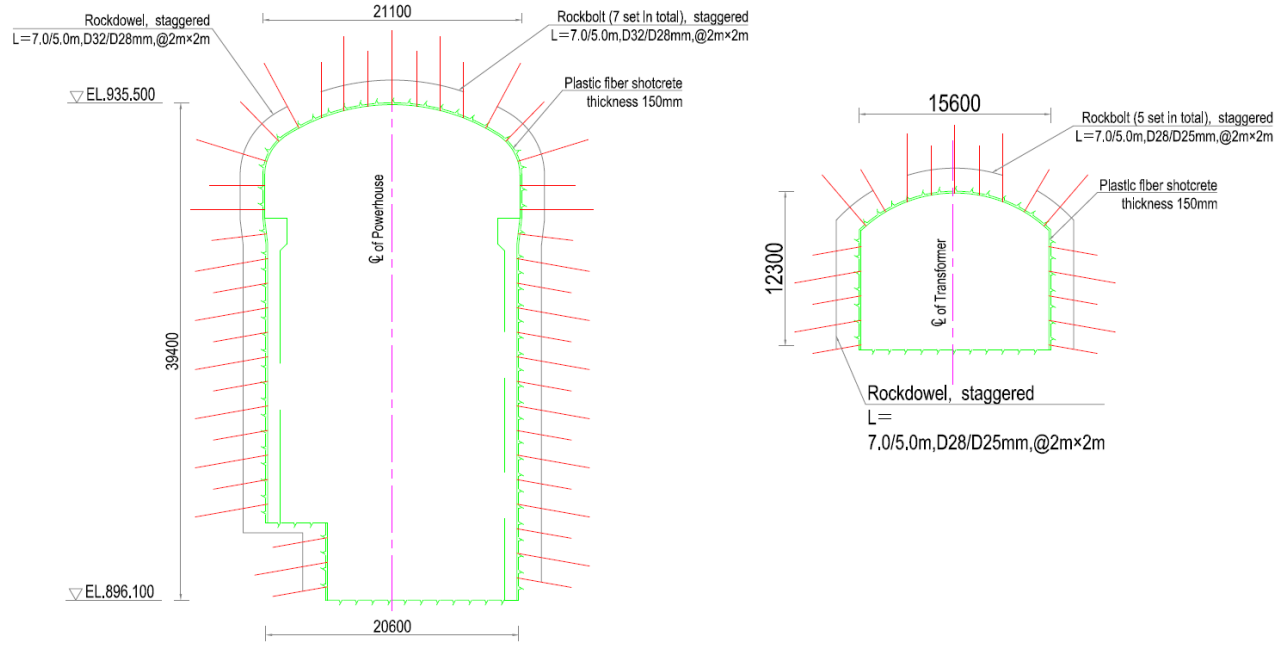


Figure 6.2-1 Prop supports of powerhouse and transformer caverns (rock type II)

Calculation result is as shown in Figures 6.2-2~6.2-4.

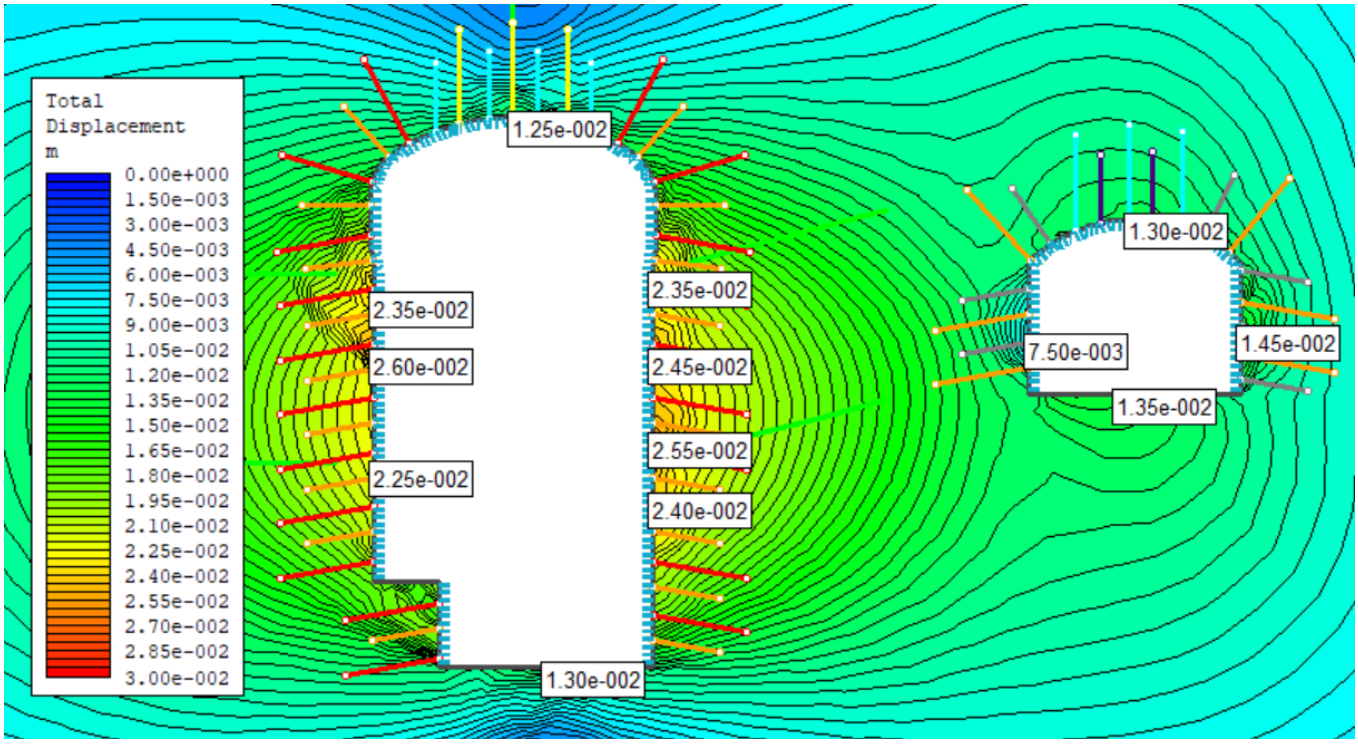


Figure 6.2-2 Total displacement after support installation (Rock Class II)

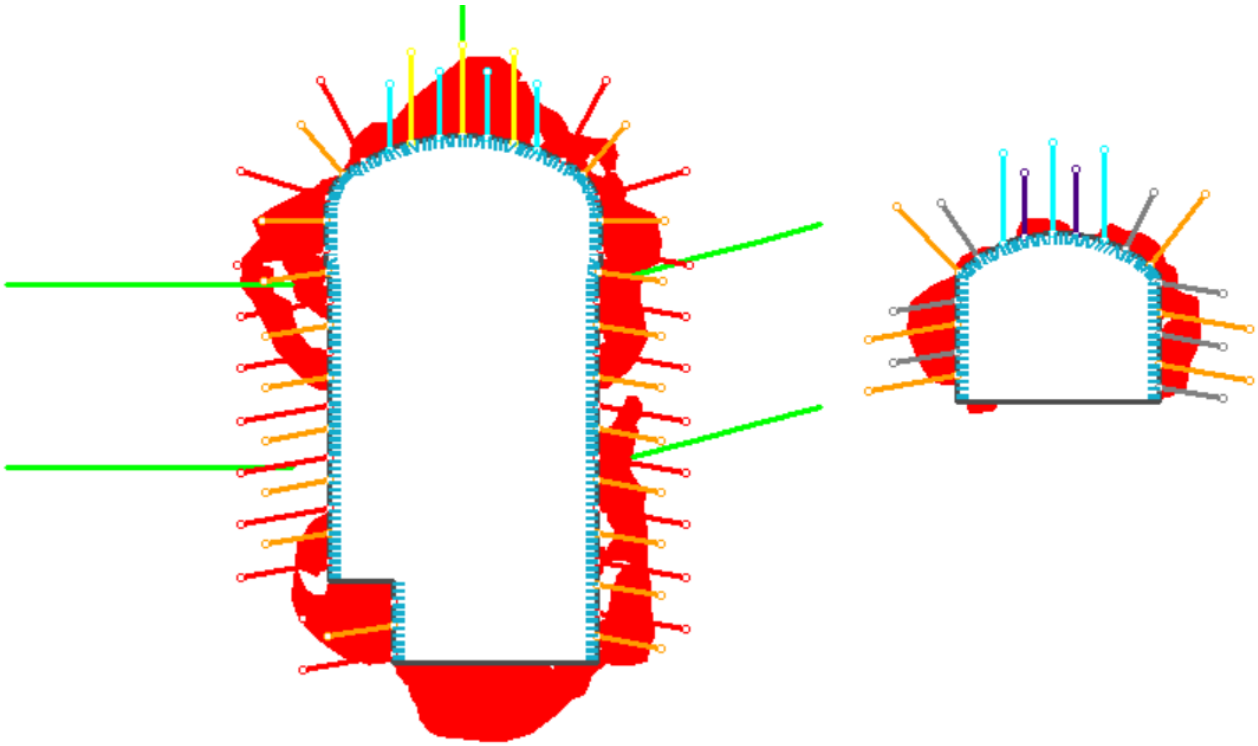


Figure 6.2-3 Plastic zone after support installation (Rock Class II)

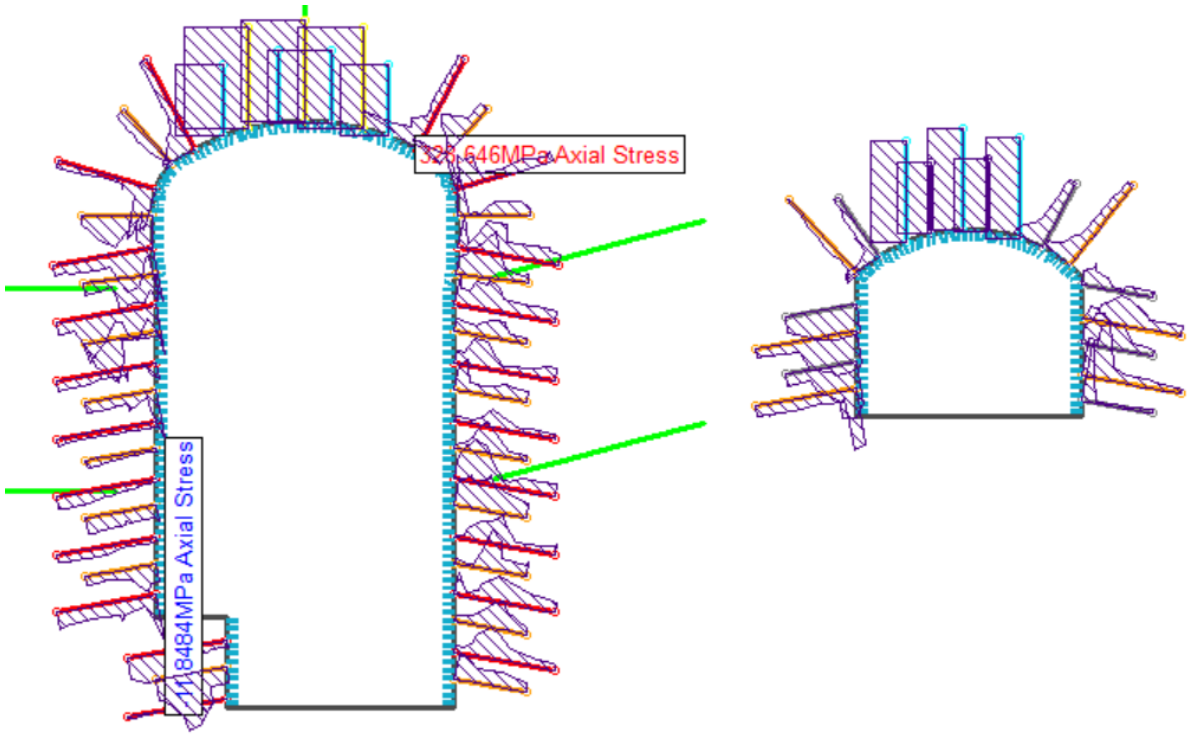


Figure 6.2-4 Axial stress of rock dowels/rock bolts after support installation (Rock Class II)

6.3 Displacement of multi-point extensometer of powerhouse cavern crown (Rock class II,prop supports)

According to FEM calculation result, displacement of monitoring points (multi-point extensometer) at each excavation stage are shown in figure 6.3-1~8. Displacement histories of each monitoring points are shown in figure 6.3-9~28.

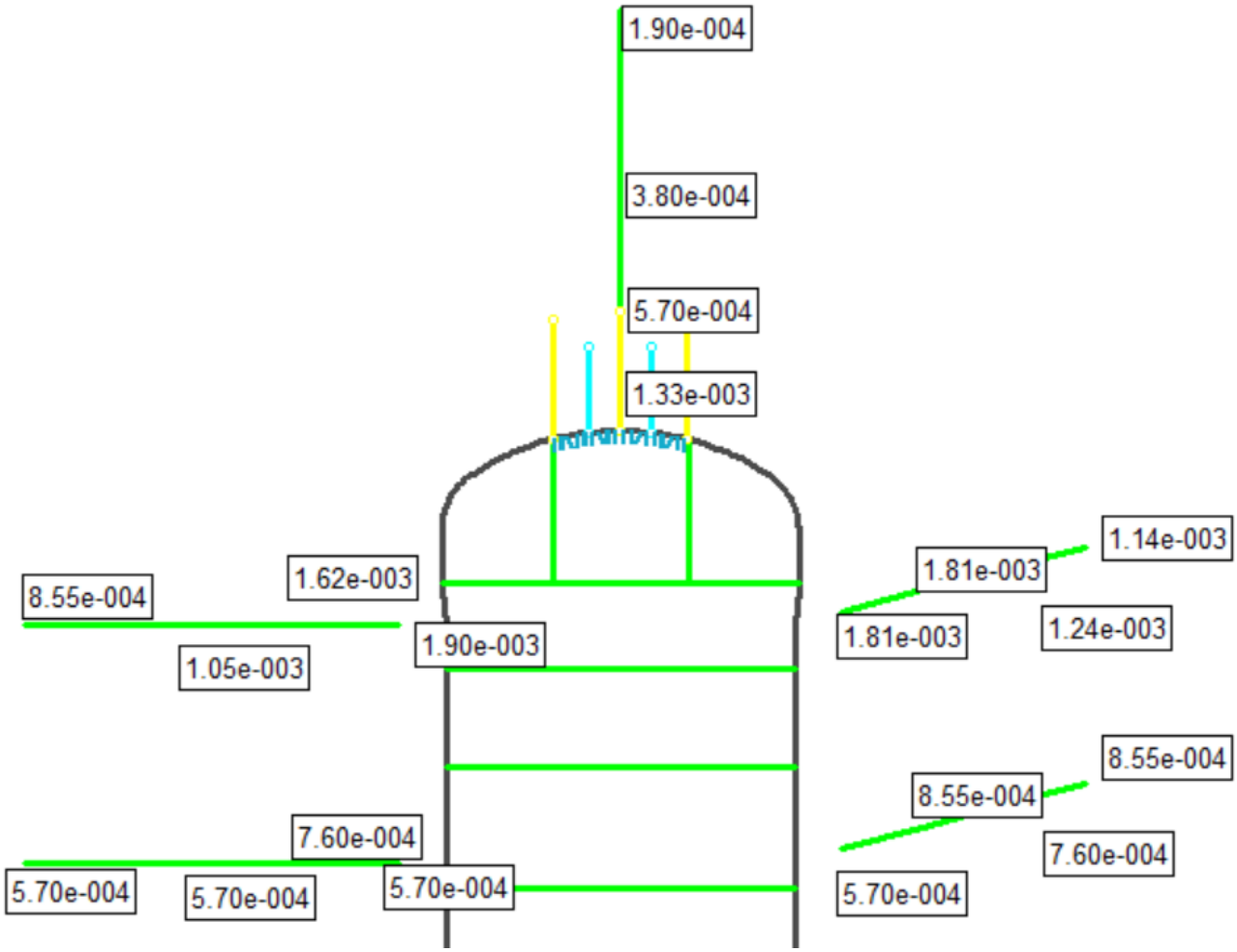


Figure 6.3-1 Displacement of monitoring points at 1st stage (Unit: m)

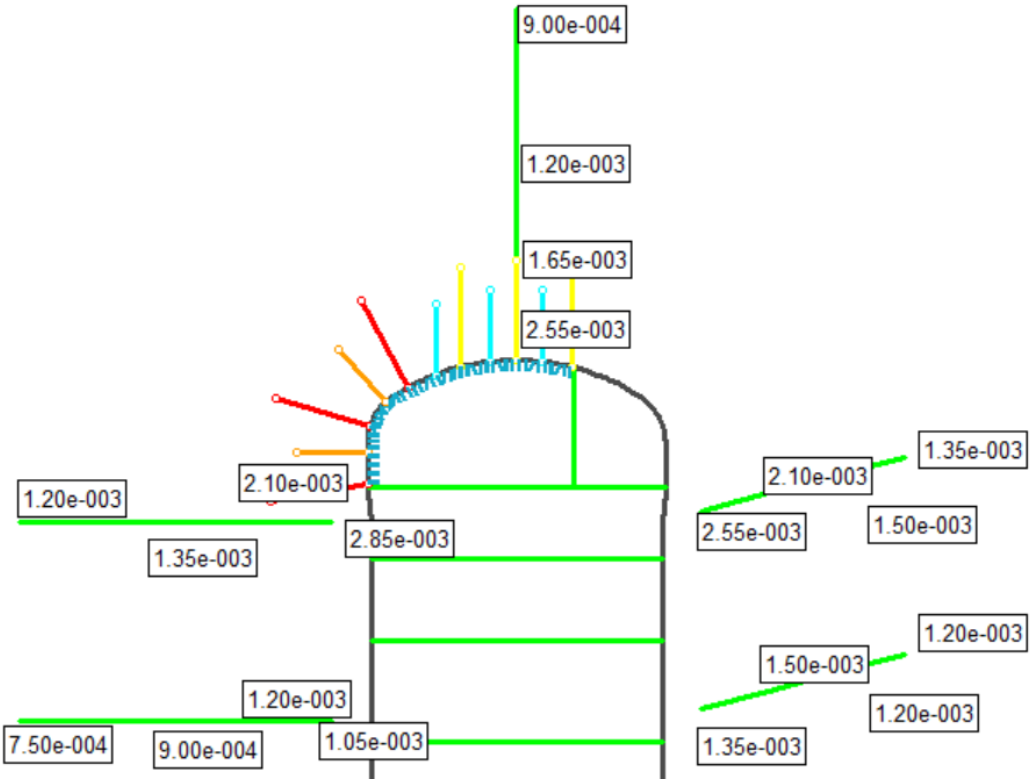


Figure 6.3-2 Displacement of monitoring points at 2nd stage (Unit: m)

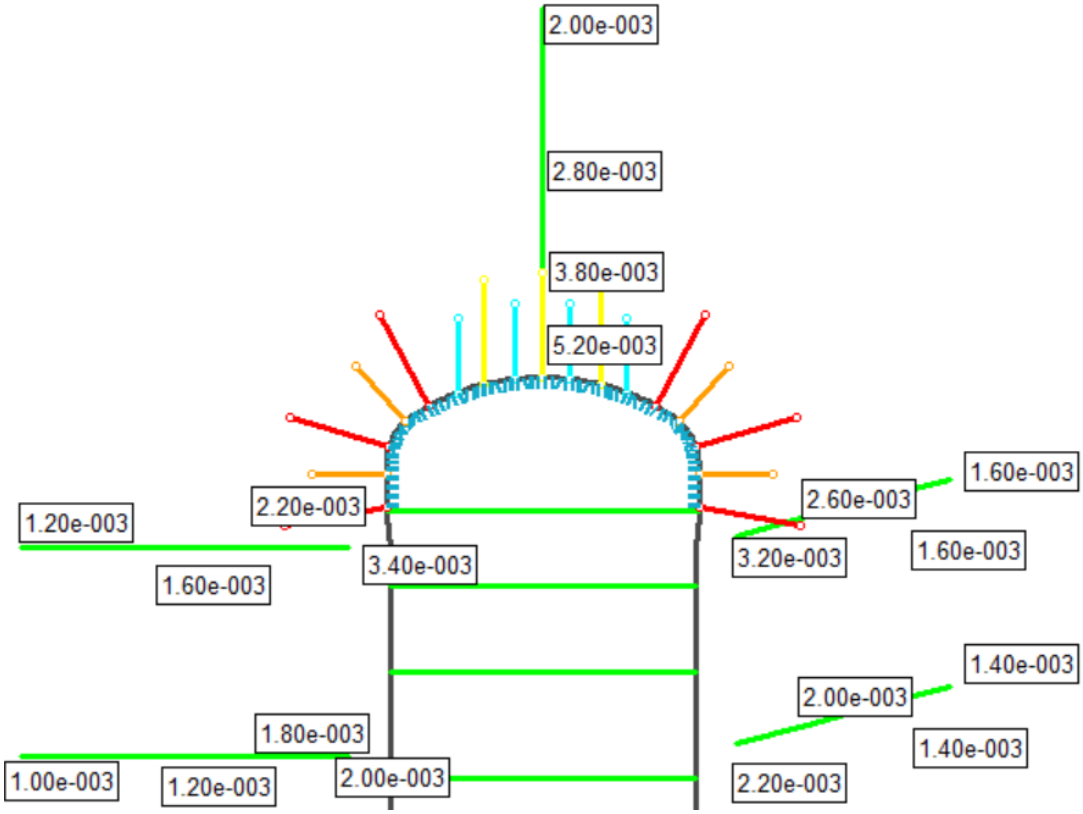


Figure 6.3-3 Displacement of monitoring points at 3rd stage (Unit: m)

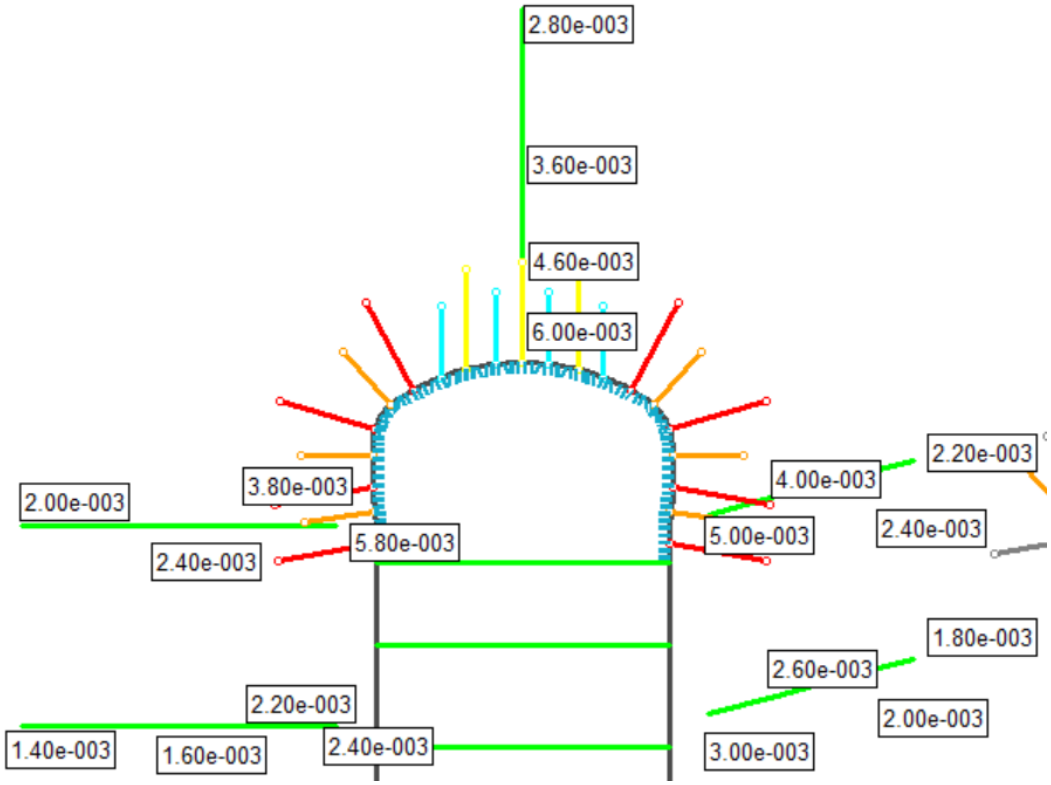


Figure 6.3-4 Displacement of monitoring points at 4th stage (Unit: m)

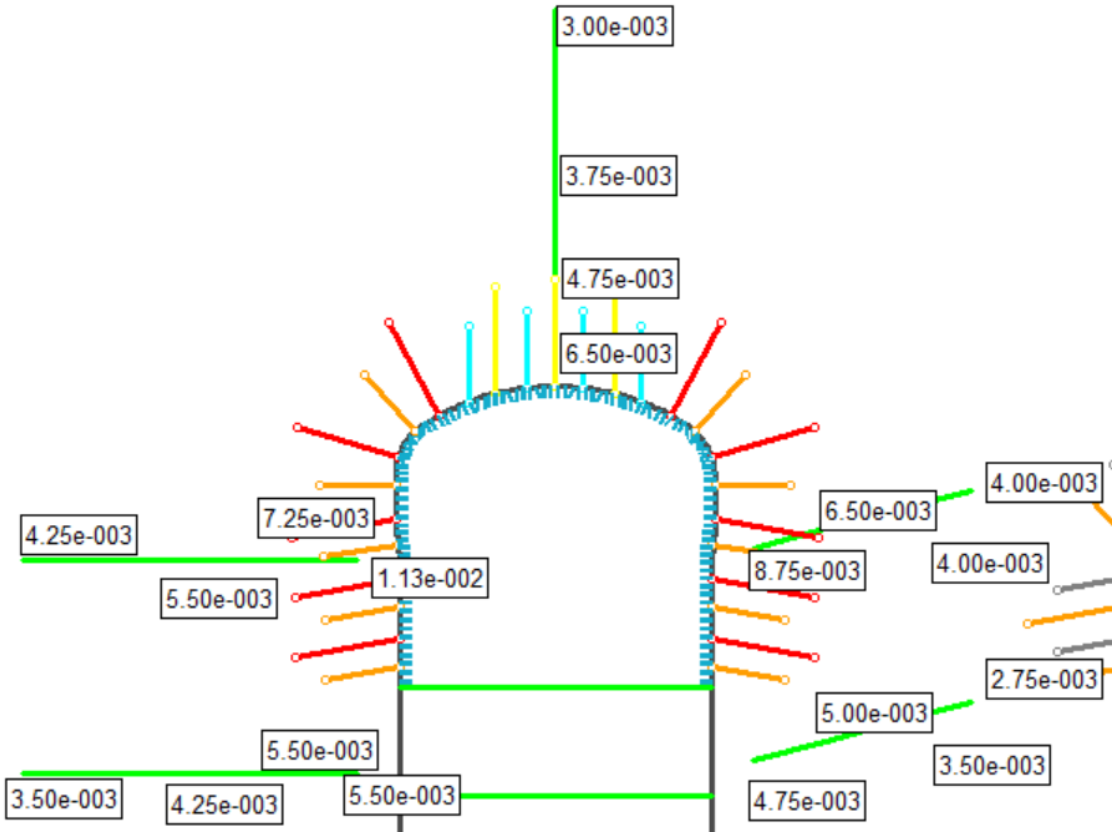


Figure 6.3-5 Displacement of monitoring points at 5th stage (Unit: m)

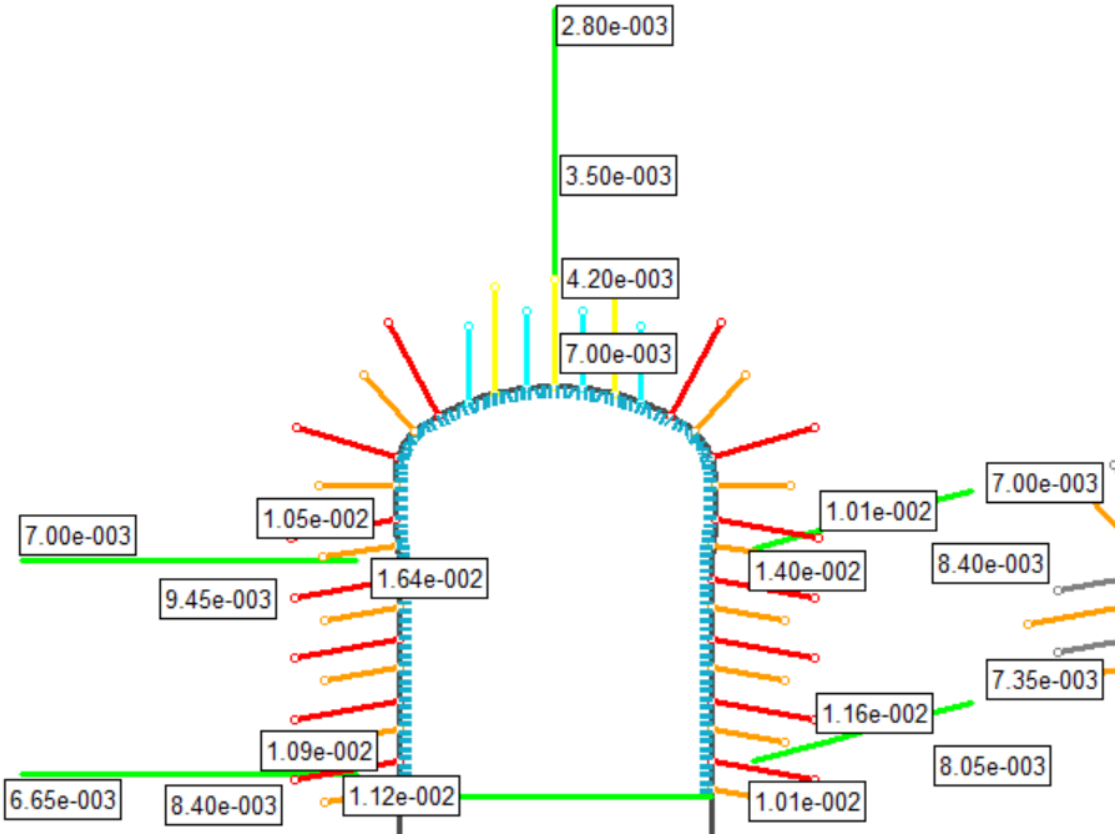


Figure 6.3-6 Displacement of monitoring points at 6th stage (Unit: m)

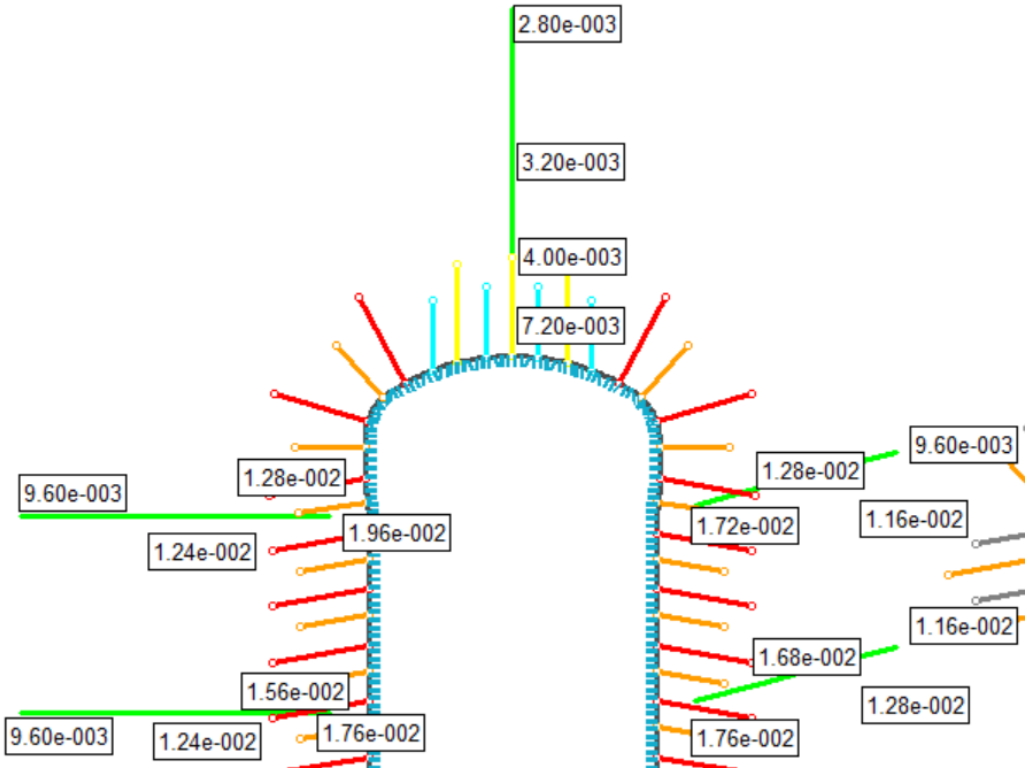


Figure 6.3-7 Displacement of monitoring points at 7th stage (Unit: m)

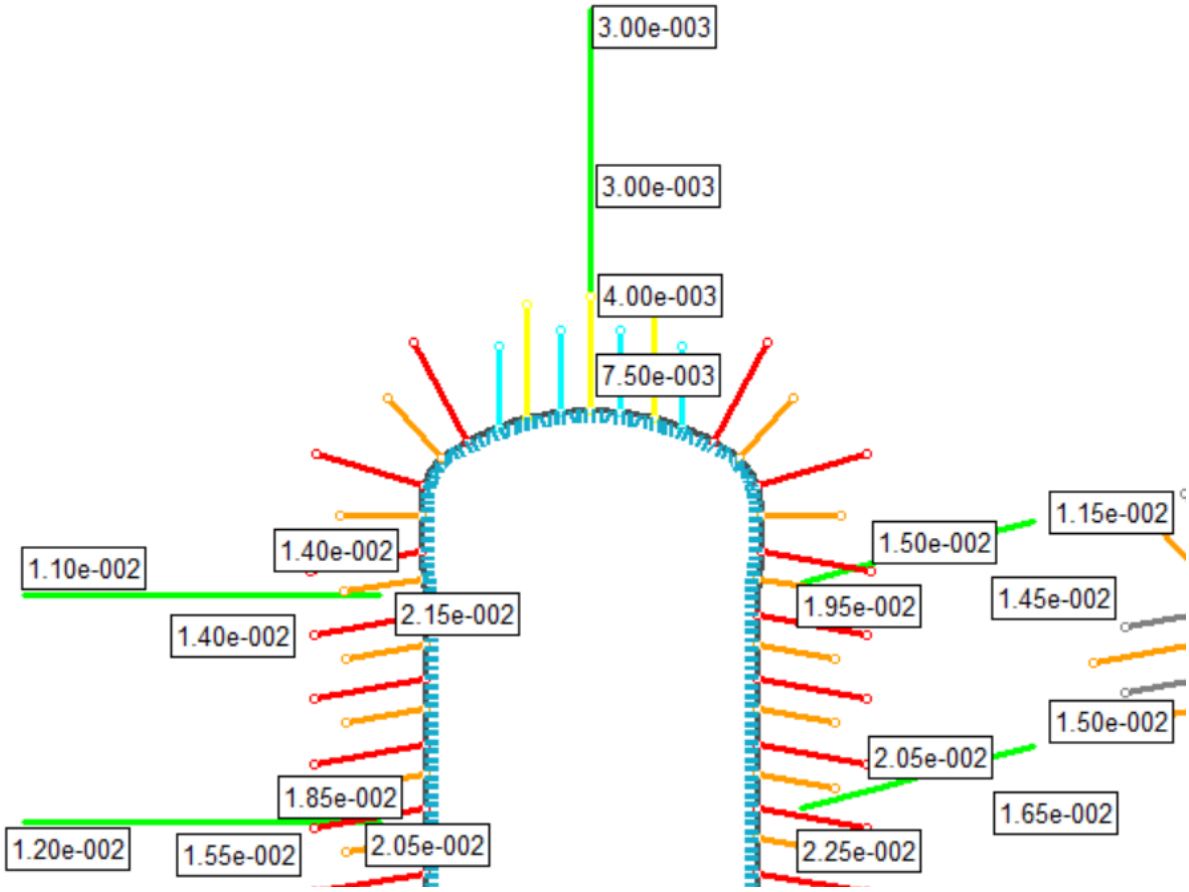


Figure 6.3-8 Displacement of monitoring points at 8th stage (Unit: m)

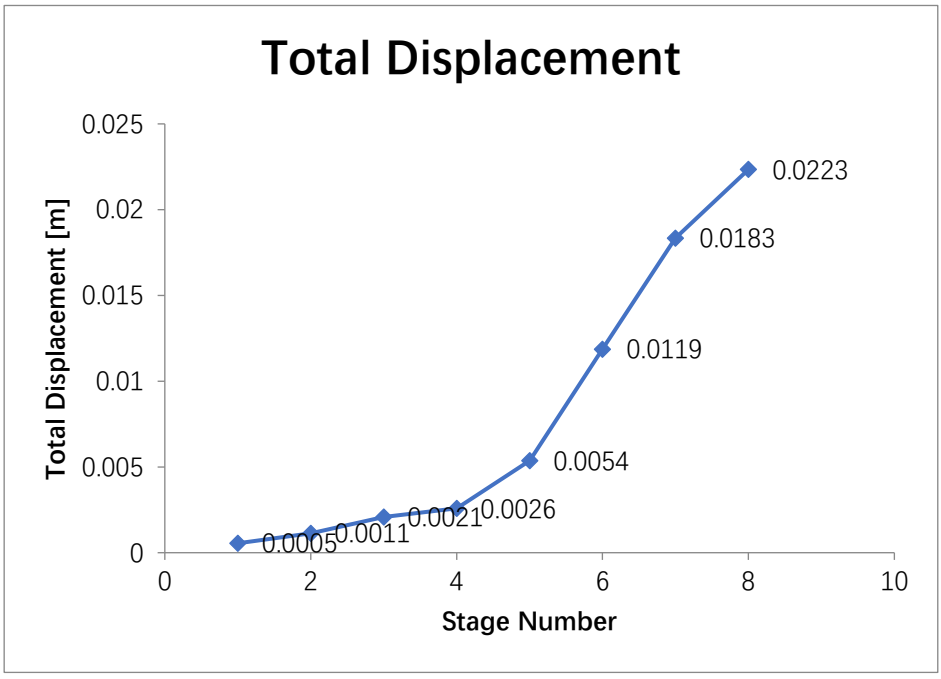


Figure 6.3-9 Displacement history of ME4-1.1 monitoring points

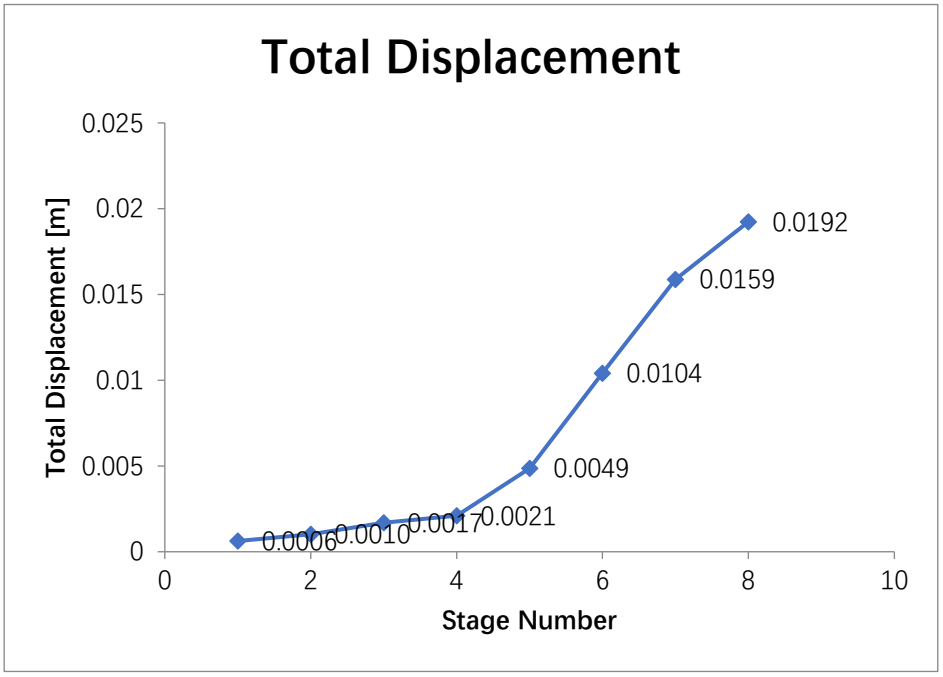


Figure 6.3-10 Displacement history of ME4-1.2 monitoring points

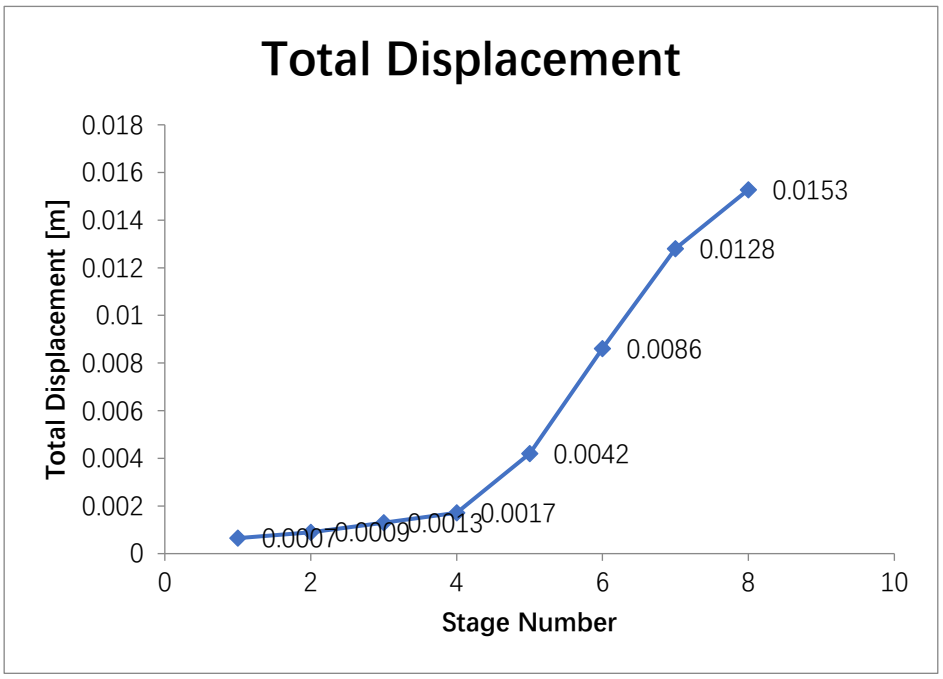


Figure 6.3-11 Displacement history of ME4-1.3 monitoring points

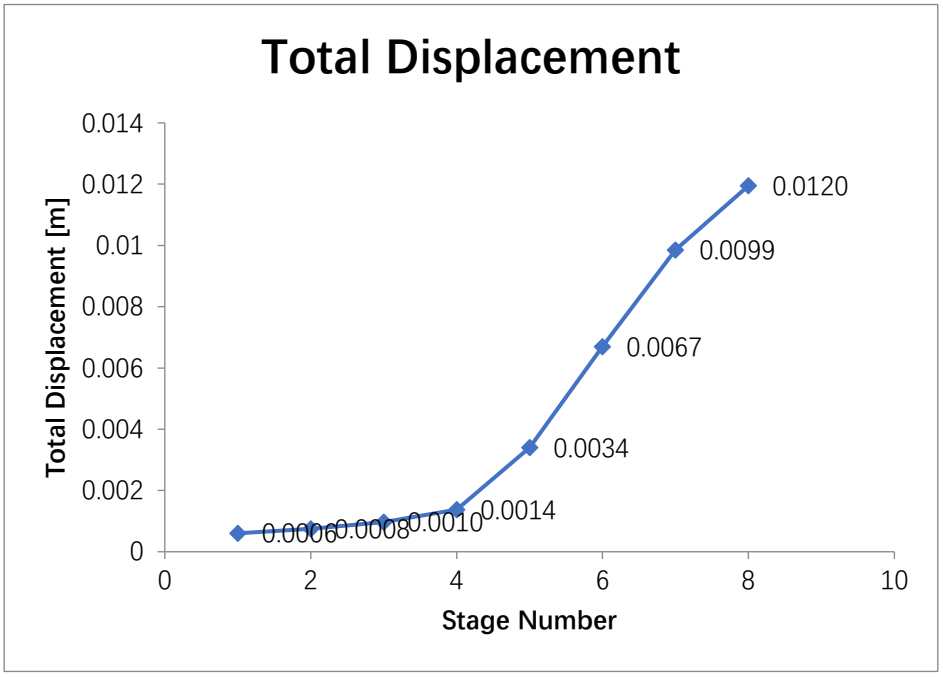


Figure 6.3-12 Displacement history of ME4-1.4 monitoring points

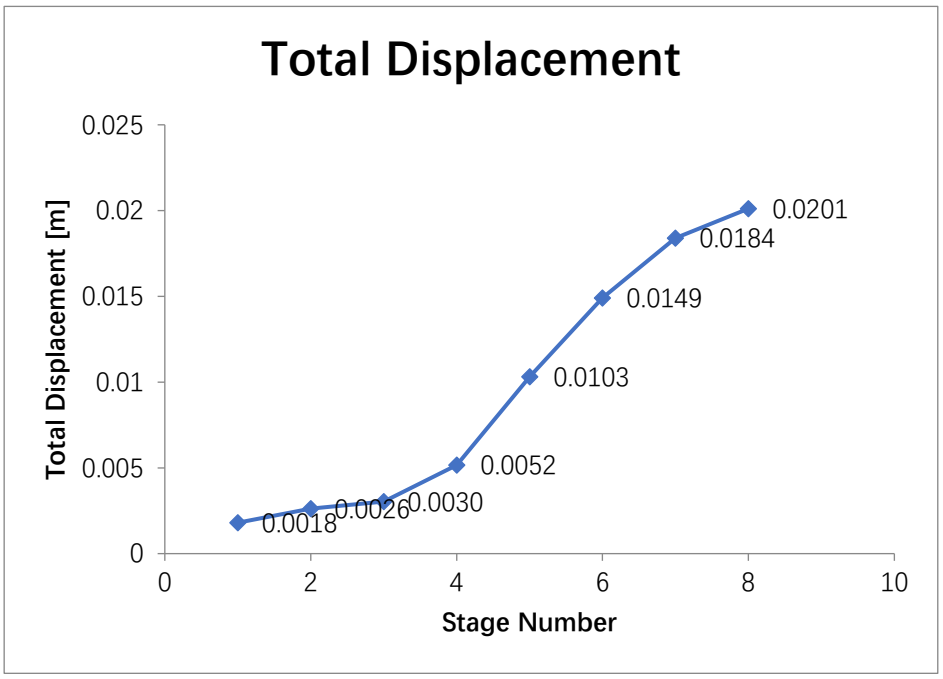


Figure 6.3-13 Displacement history of ME4-2.1 monitoring points

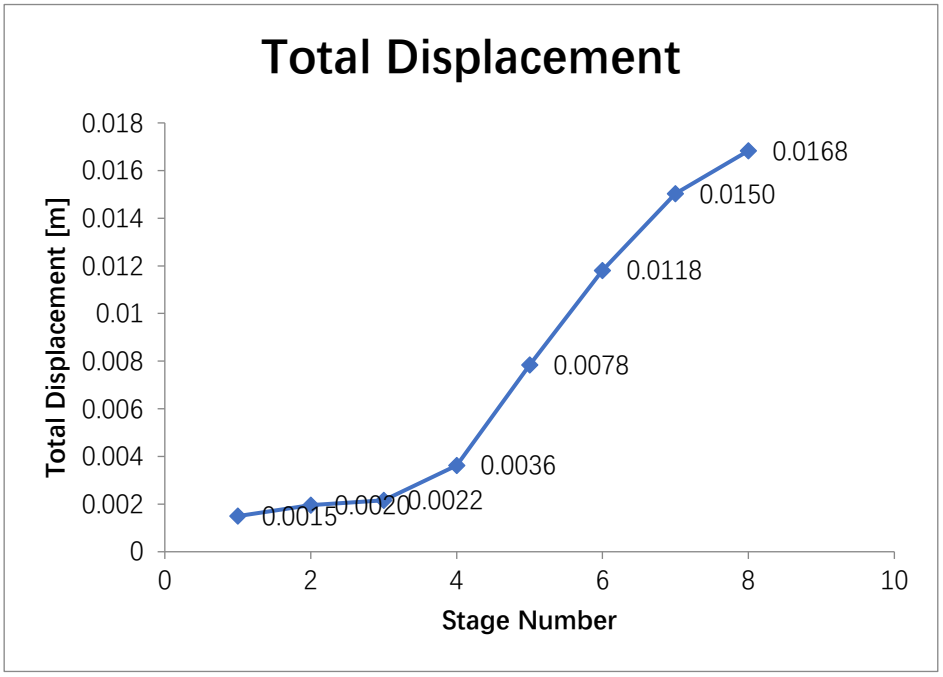


Figure 6.3-14 Displacement history of ME4-2.2 monitoring points

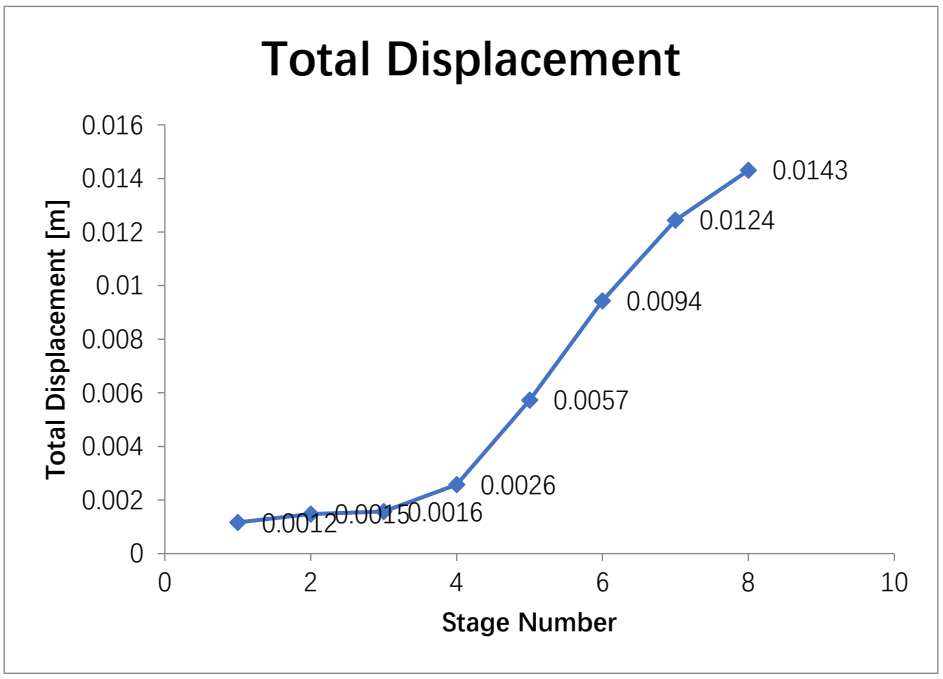


Figure 6.3-15 Displacement history of ME4-2.3 monitoring points

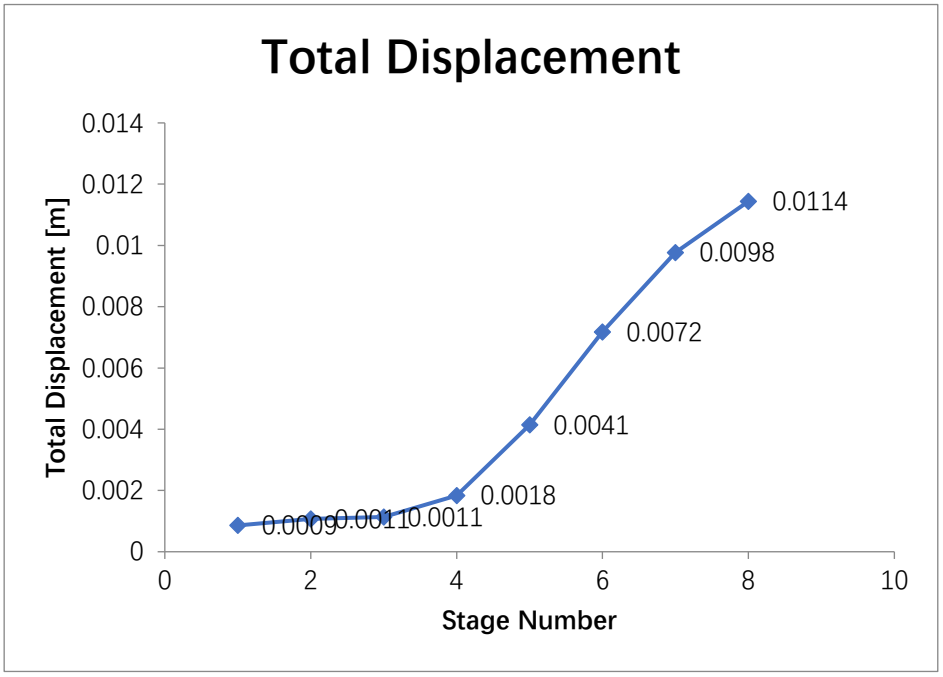


Figure 6.3-16 Displacement history of ME4-2.4 monitoring points

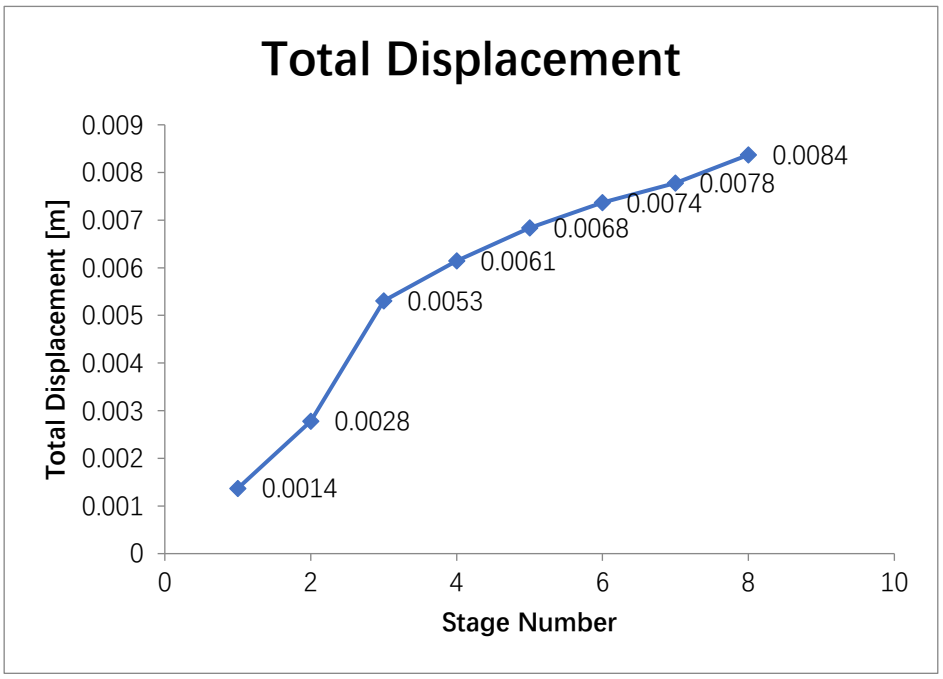


Figure 6.3-17 Displacement history of ME4-3.1 monitoring points

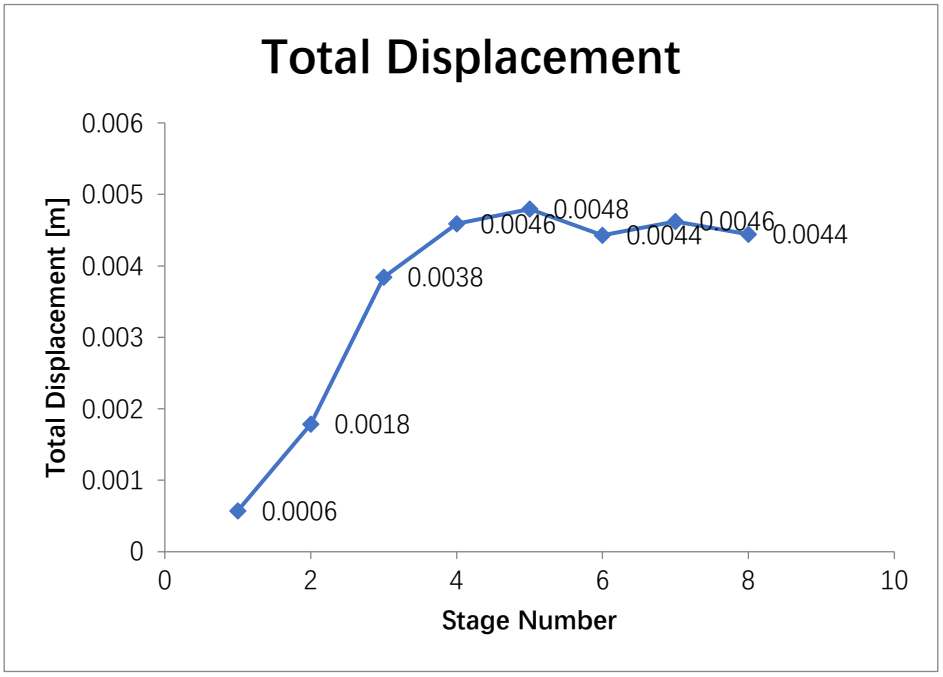


Figure 6.3-18 Displacement history of ME4-3.2 monitoring points

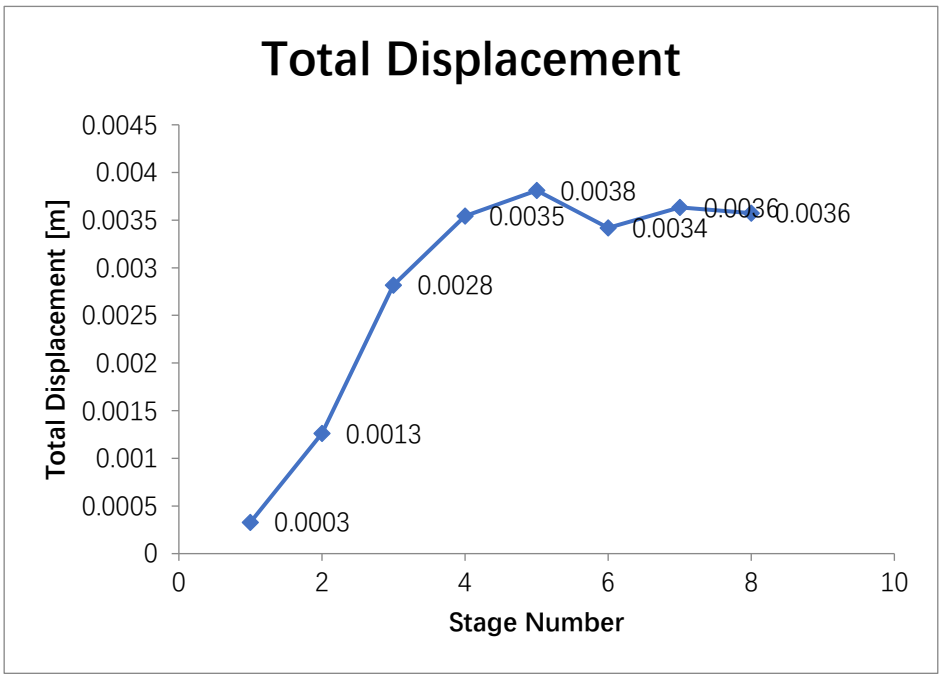


Figure 6.3-19 Displacement history of ME4-3.3 monitoring points

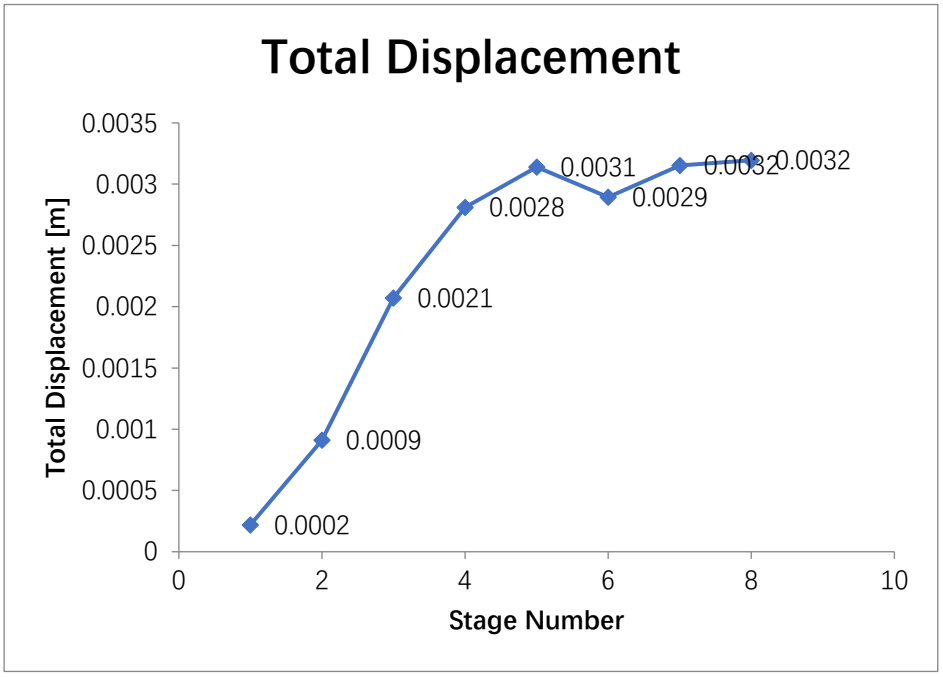


Figure 6.3-20 Displacement history of ME4-3.4 monitoring points

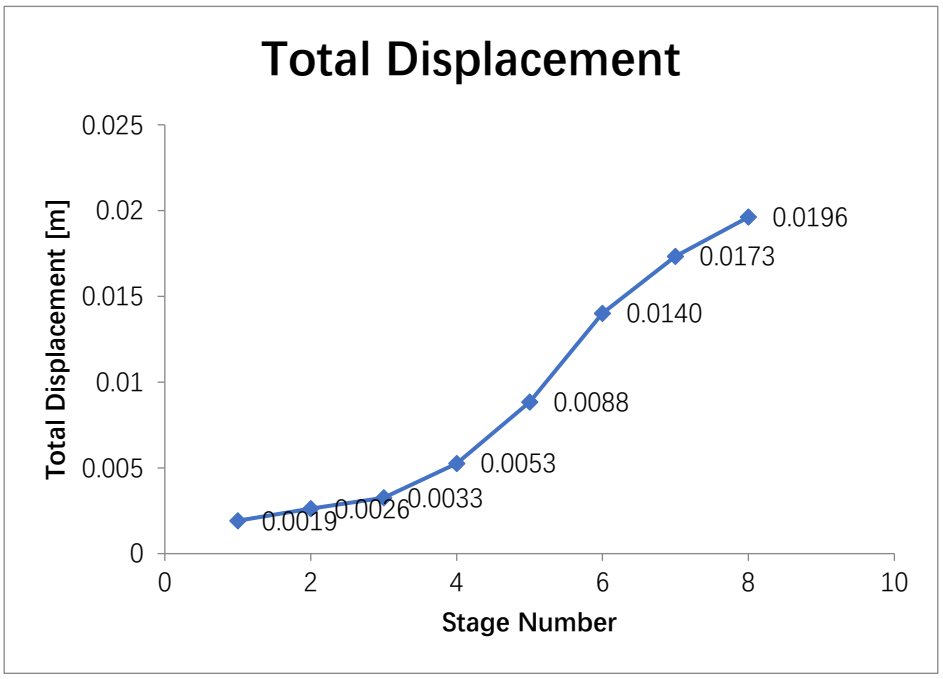


Figure 6.3-21 Displacement history of ME4-4.1 monitoring points

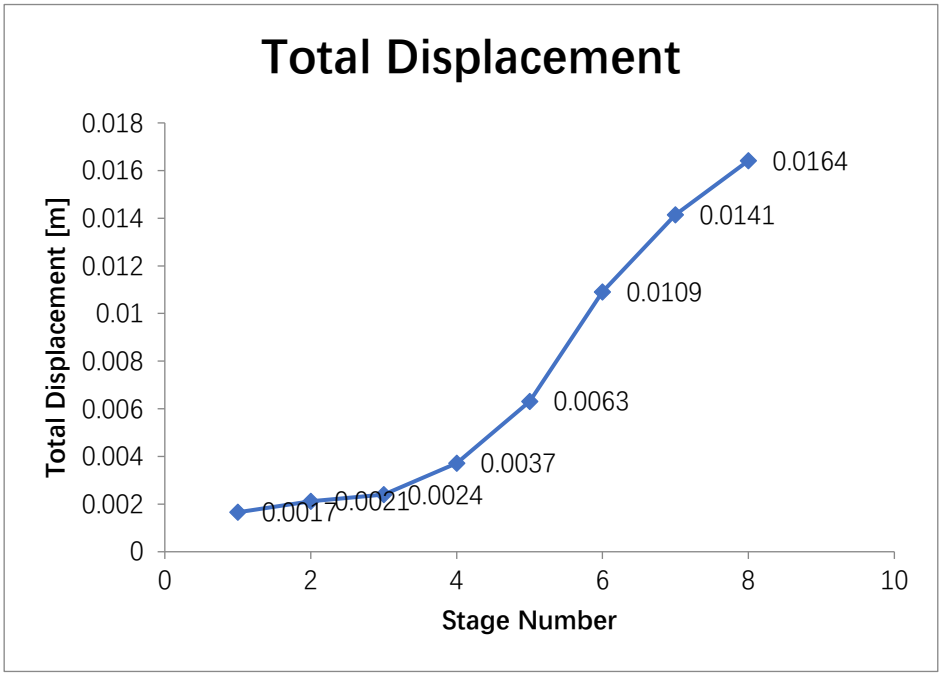


Figure 6.3-22 Displacement history of ME4-4.2 monitoring points

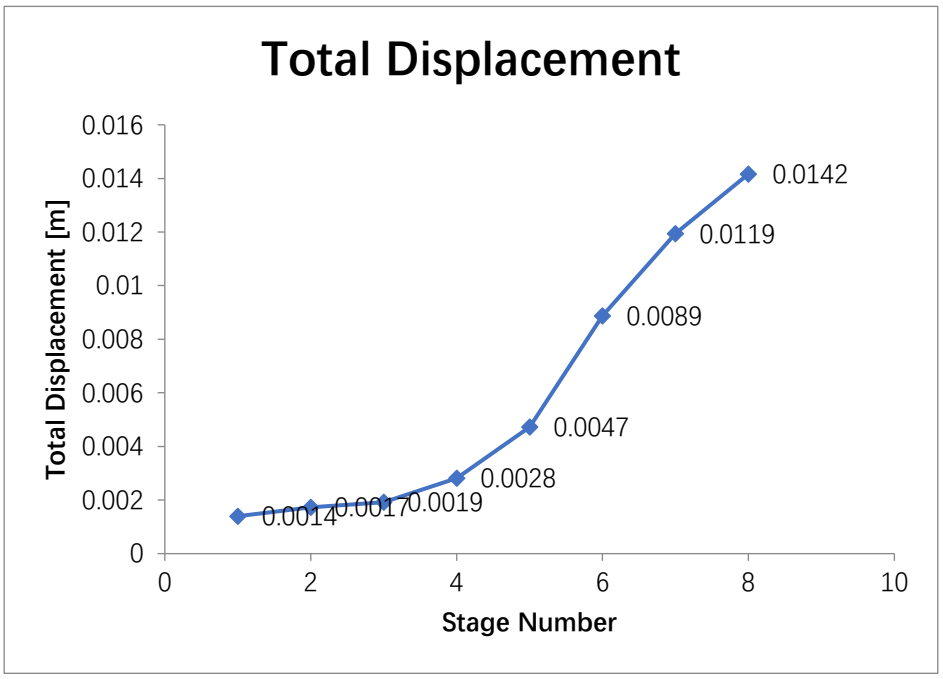


Figure 6.3-23 Displacement history of ME4-4.3 monitoring points

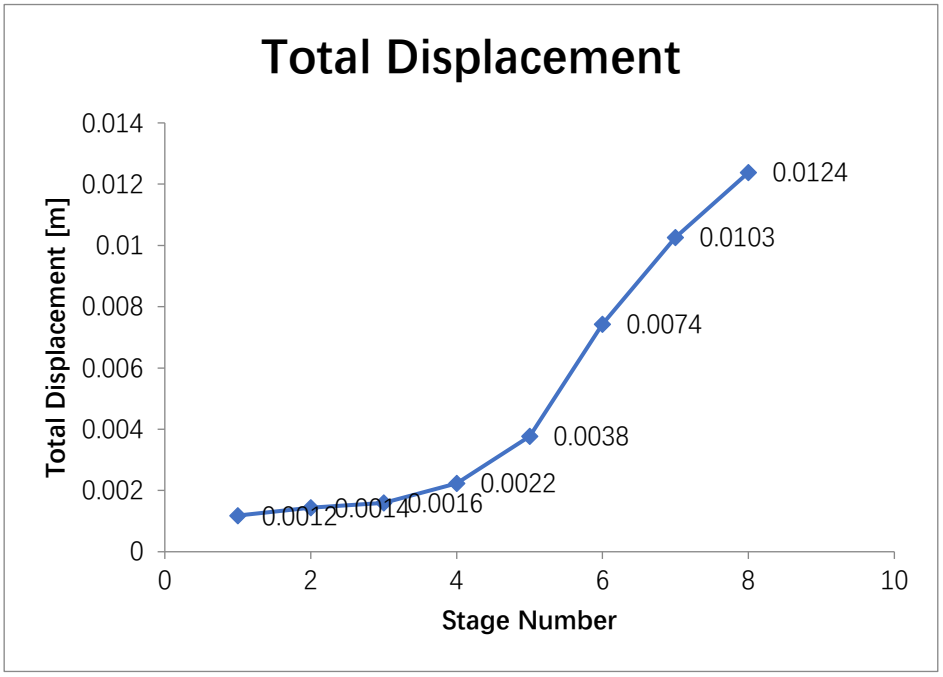


Figure 6.3-24 Displacement history of ME4-4.4 monitoring points

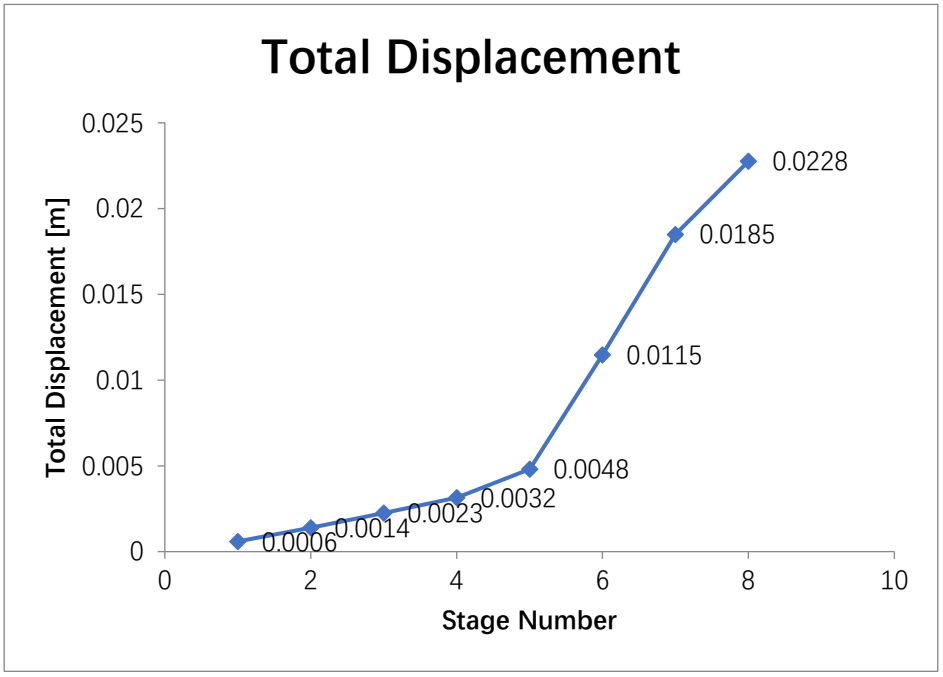


Figure 6.3-25 Displacement history of ME4-5.1 monitoring points

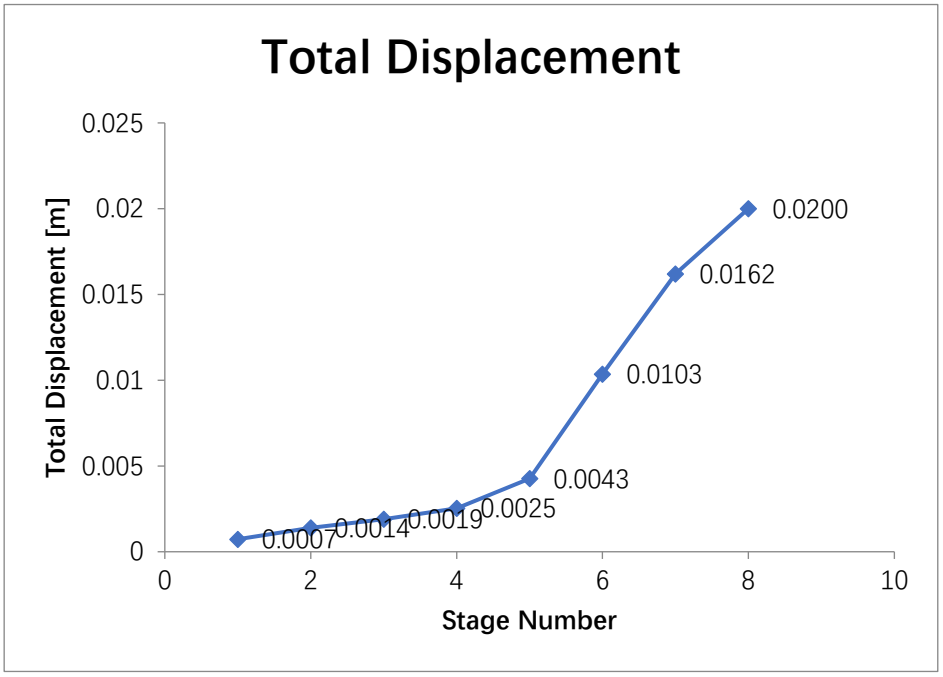


Figure 6.3-26 Displacement history of ME4-5.2 monitoring points

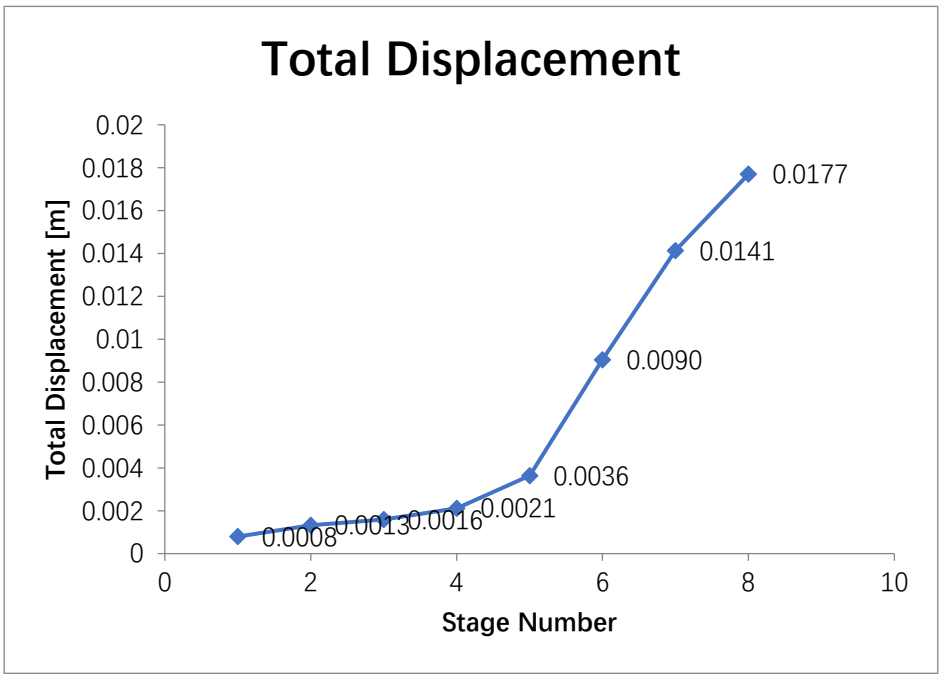


Figure 6.3-27 Displacement history of ME4-5.3 monitoring points

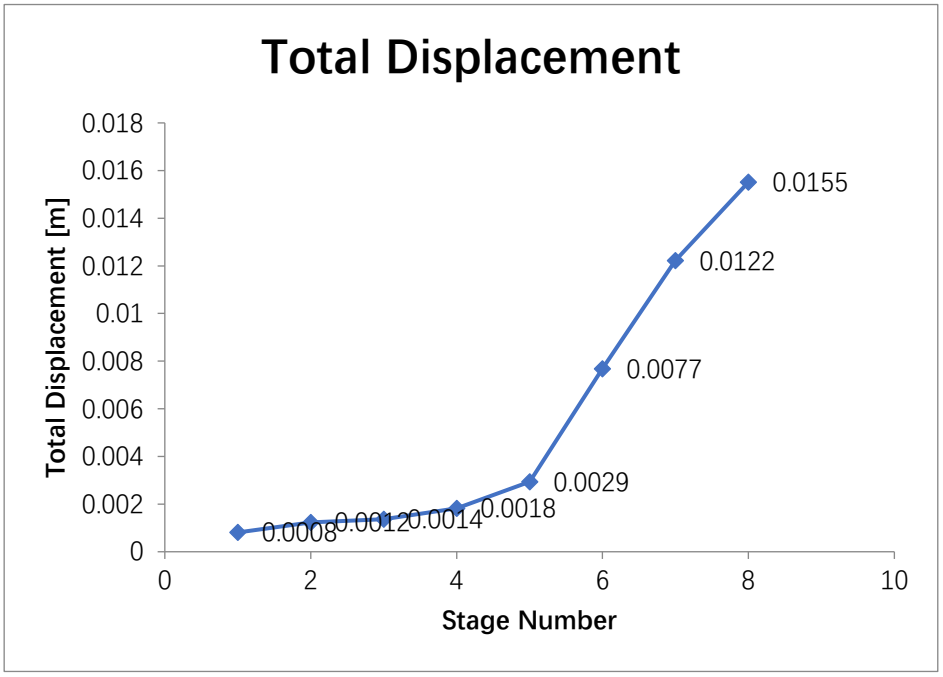


Figure 6.3-28 Displacement history of ME4-5.4 monitoring points

6.4 Rock dowels/bolts stress (prop supports)

According to calculation result, the stress of typical rock dowels/bolts are shown as follows.

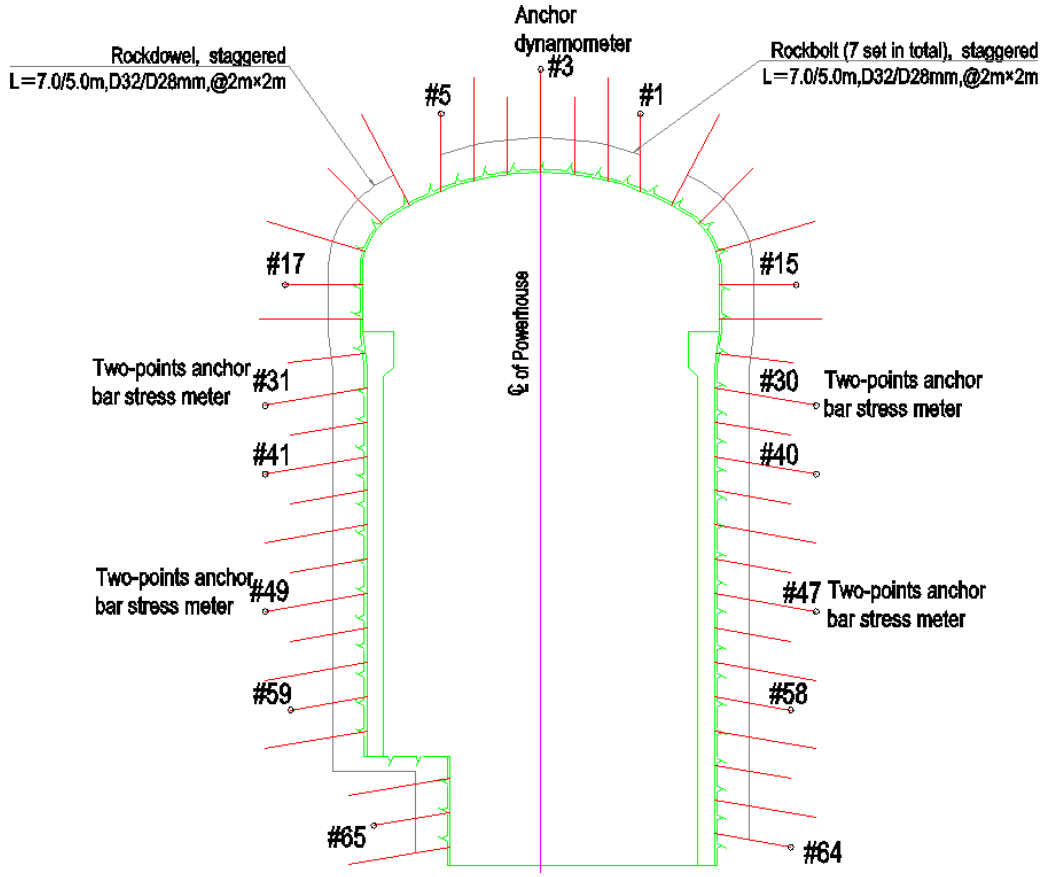


Figure 6.4-1 Typical rock dowels/bolts sketch

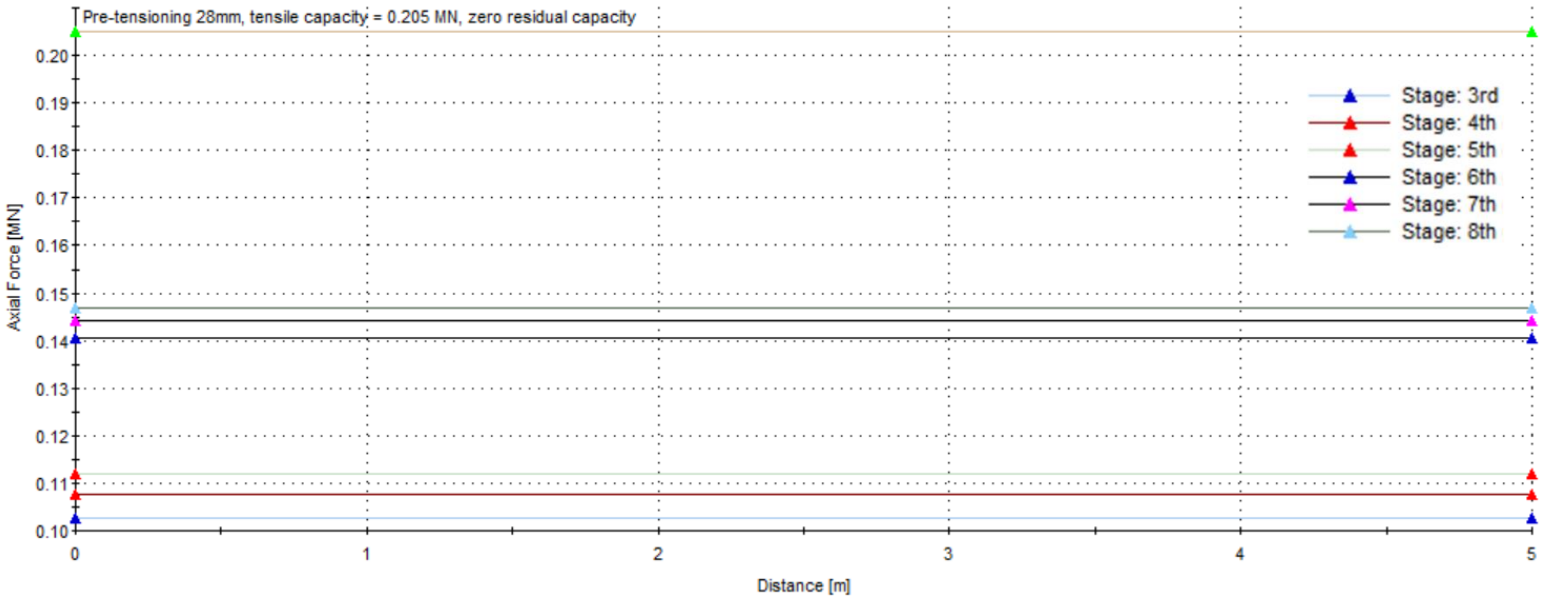
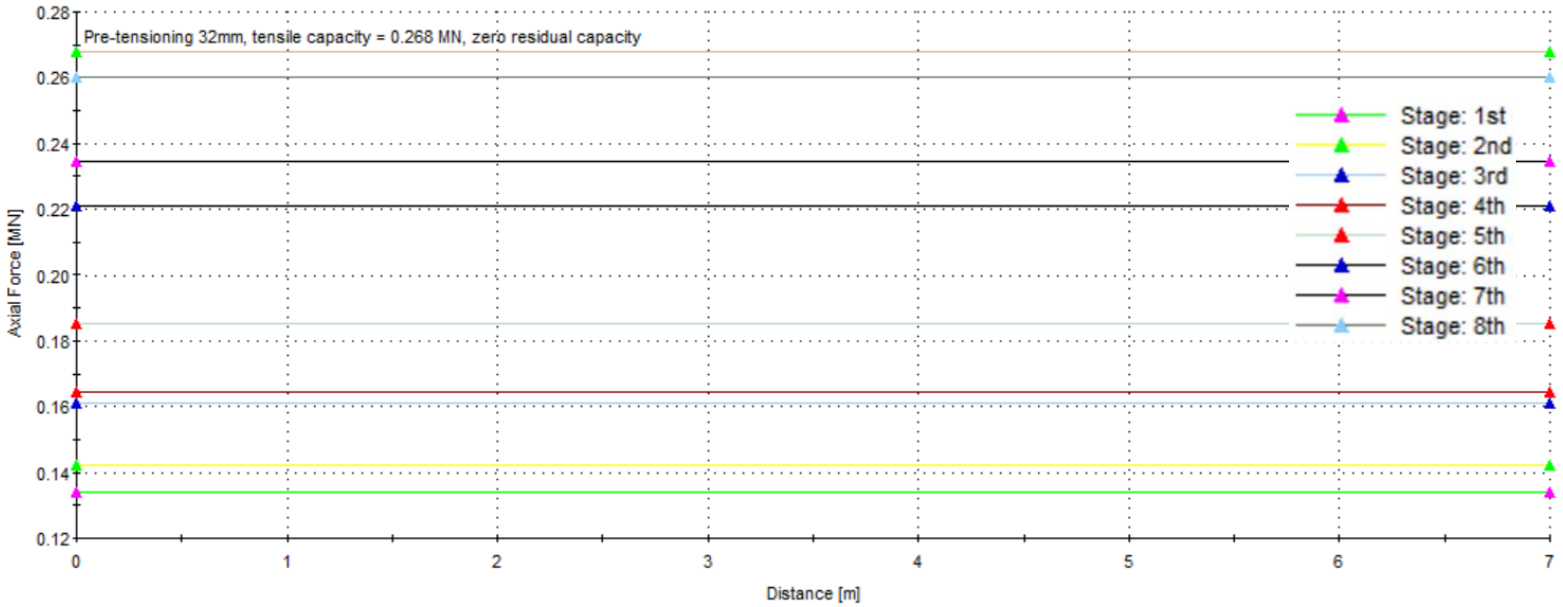


Figure 6.4-2 Axial force of rock bolt #1 developing with excavation stages



Excavation stage	1st	2nd	3rd	4th	5th	6th	7th	8th
Axial force(MN)	0.134	0.142	0.161	0.164	0.185	0.221	0.235	0.260

Figure 6.4-3 Axial force of rock bolt #3 developing with excavation stages(Anchor dynamometer)

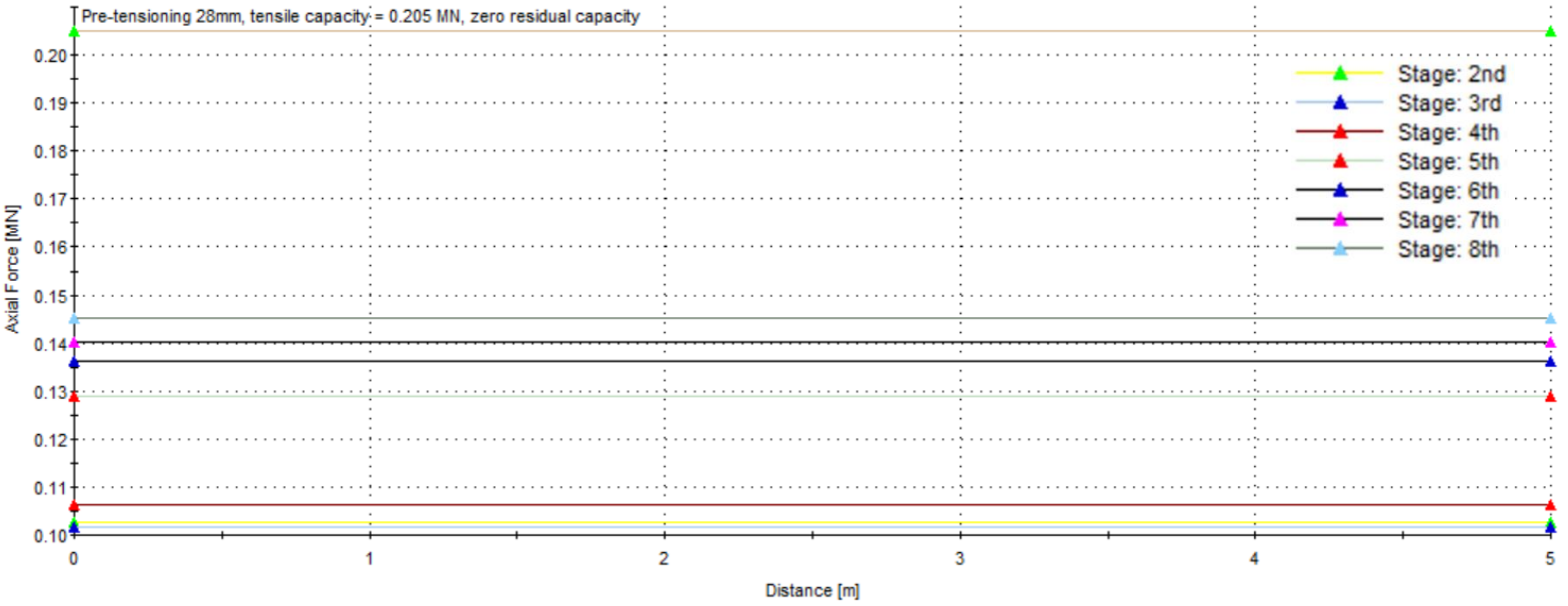


Figure 6.4-4 Axial force of rock bolt #5 developing with excavation stages

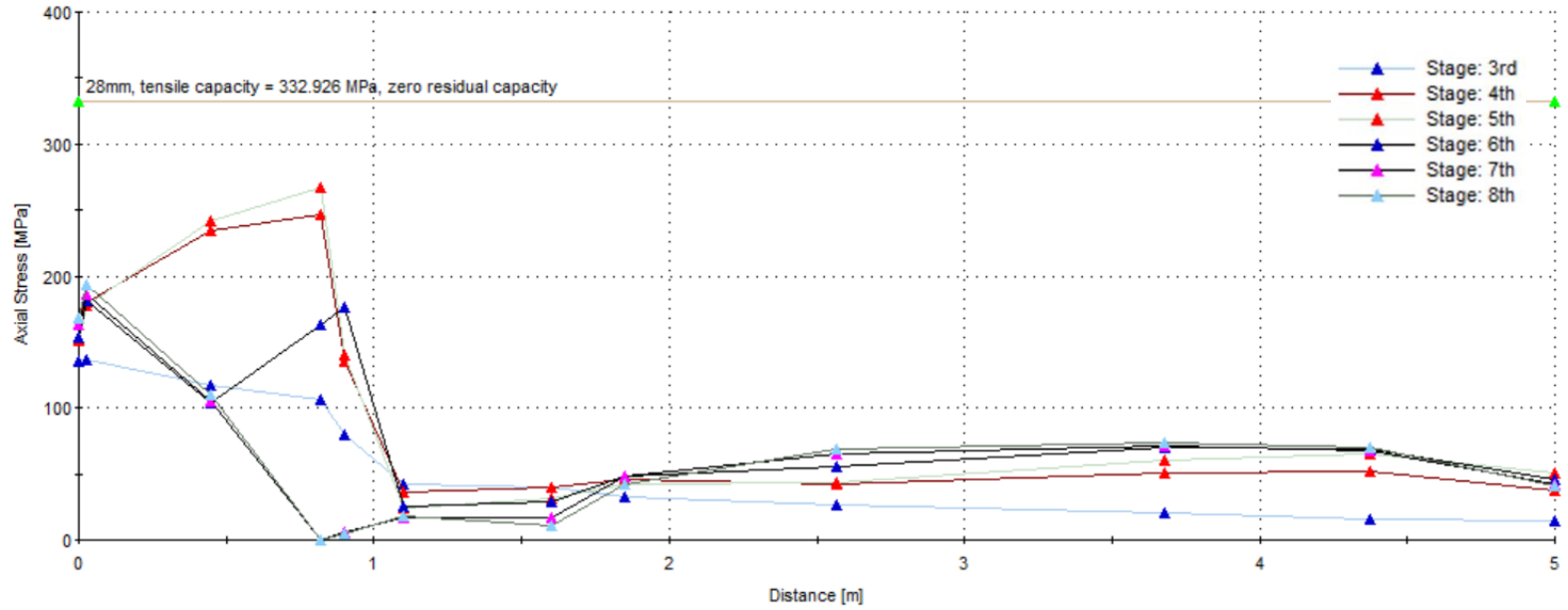


Figure 6.4-5 Axial stress of rock dowel #15 developing with excavation stages

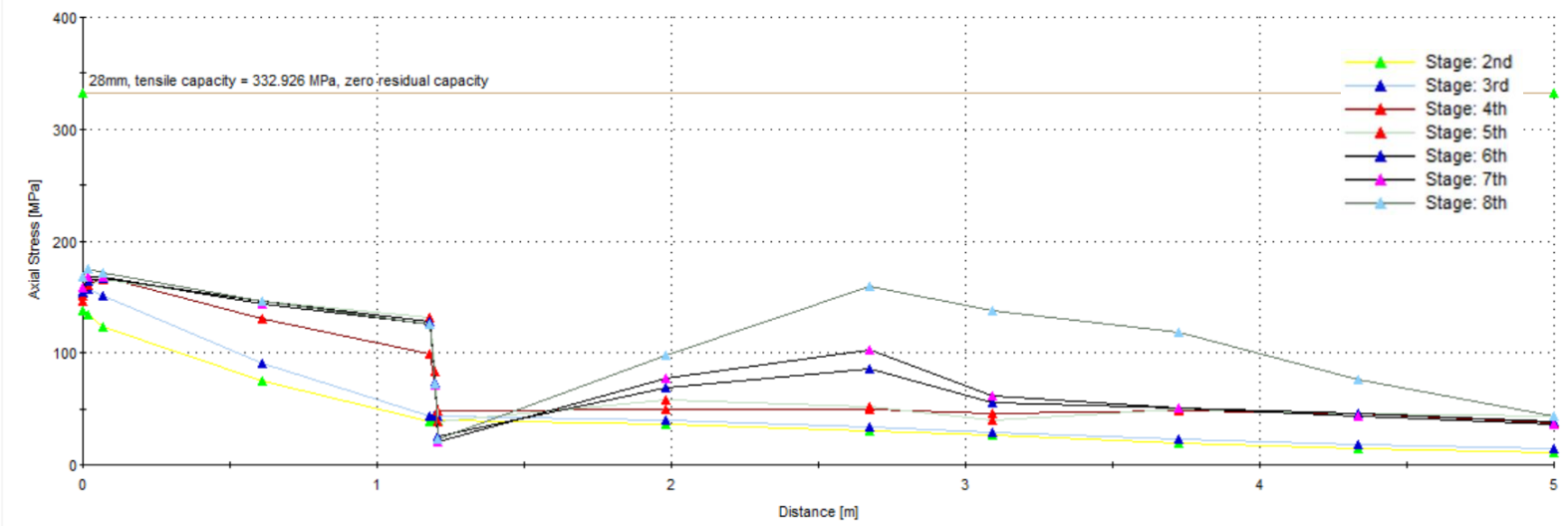
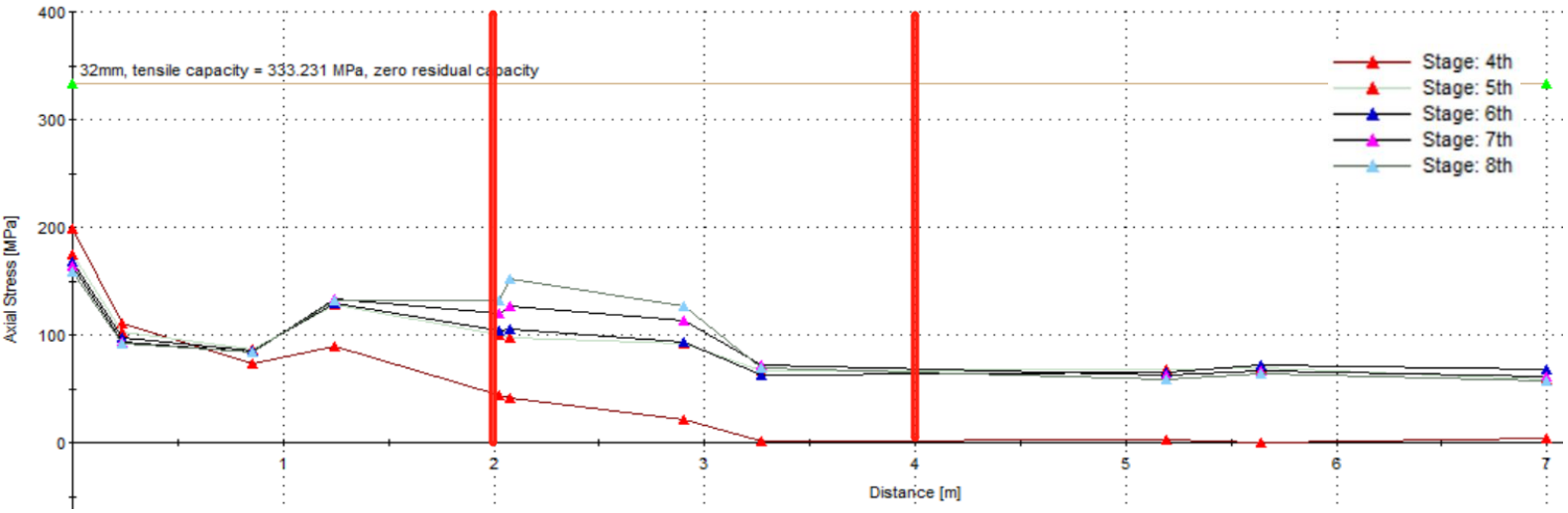


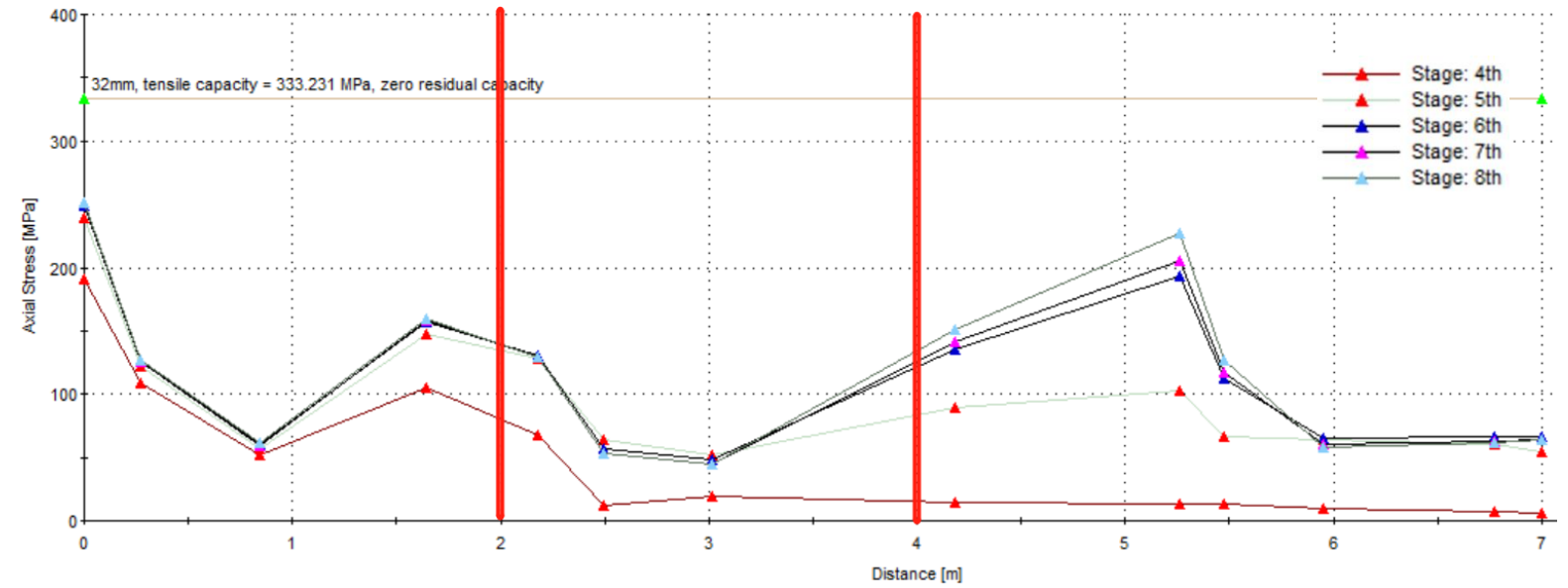
Figure 6.4-6 Axial stress of rock dowel #17 developing with excavation stages



Excavation stage	4th	5th	6th	7th	8th
Axial stress of 2m distance(MPa)	45.69	100.40	105.24	120.58	131.95
Axial stress of 4m distance (MPa)	1.96	67.64	64.24	68.44	65.33

Figure 6.4-7 Axial stress of rock dowel #30 developing with excavation stages

(Two-points anchor bar stress meter)



Excavation stage	4th	5th	6th	7th	8th
Axial stress of 2m distance(MPa)	80.70	135.11	139.18	138.94	139.88
Axial stress of 4m distance(MPa)	15.67	83.25	121.64	125.53	134.25

Figure 6.4-8 Axial stress of rock dowel #31 developing with excavation stages

(Two-points anchor bar stress meter)

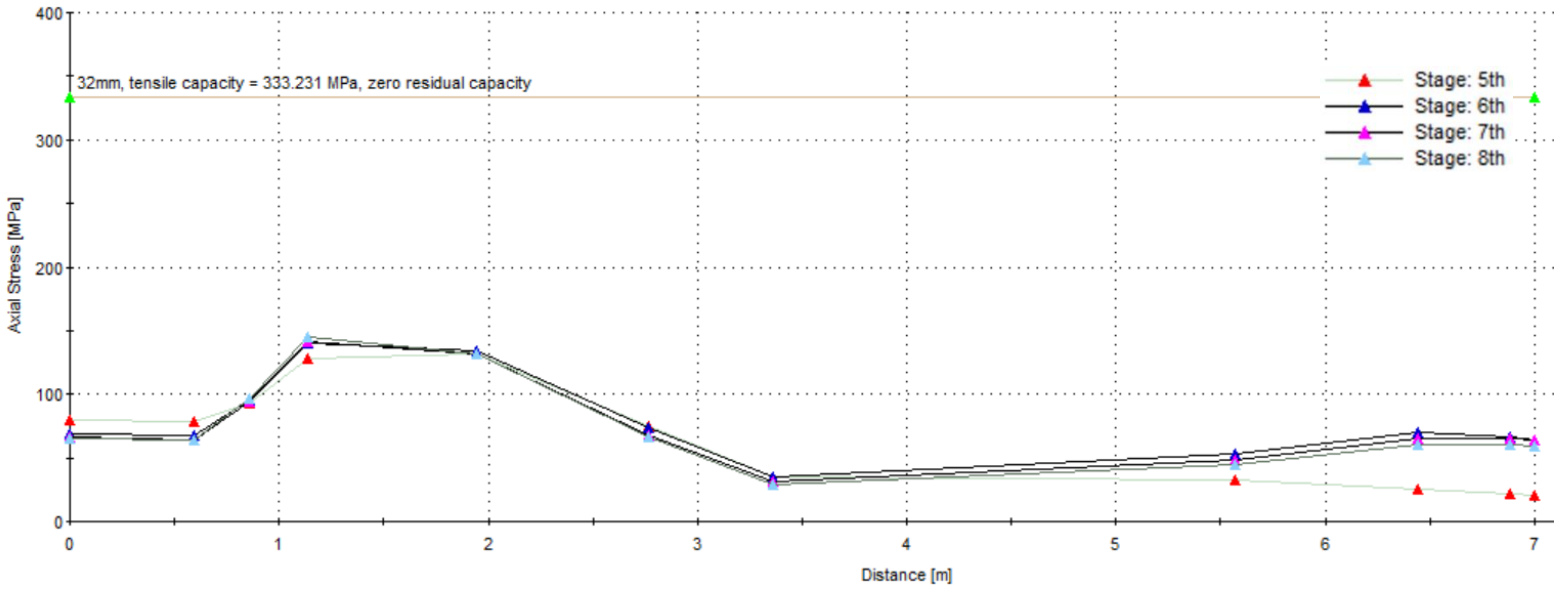


Figure 6.4-9 Axial stress of rock dowel #40 developing with excavation stages

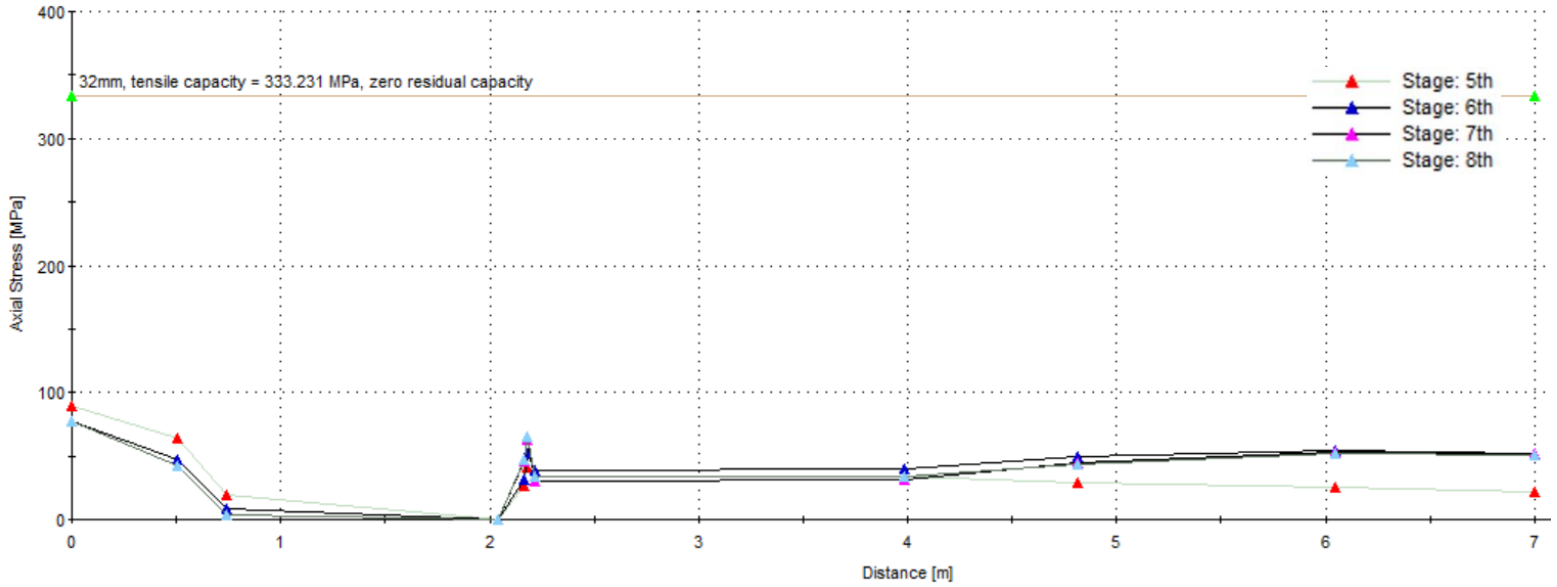
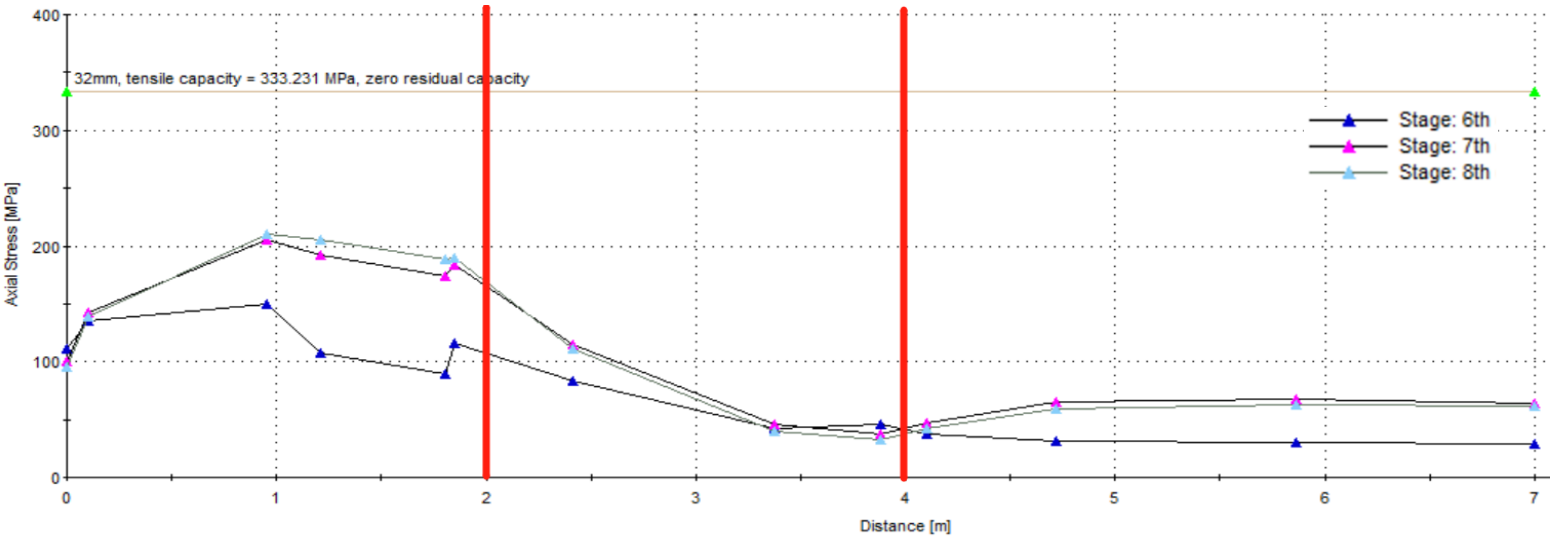


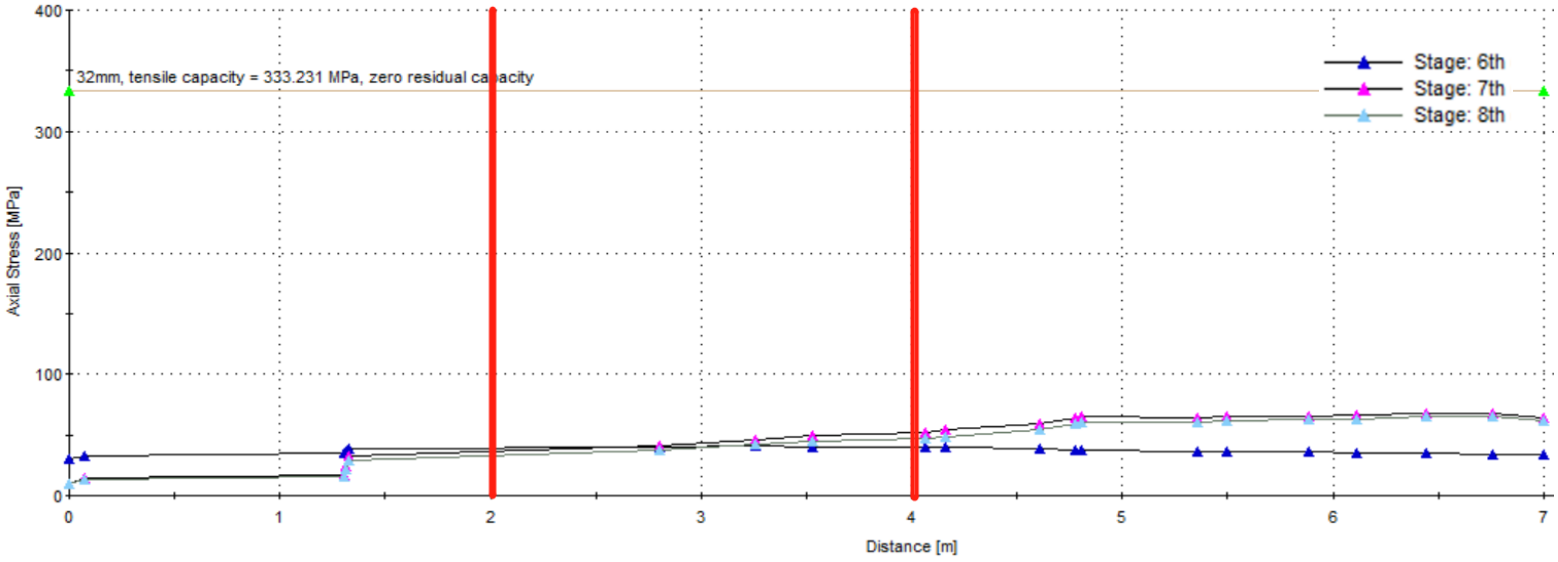
Figure 6.4-10 Axial stress of rock dowel #41 developing with excavation stage



Excavation stage	6th	7th	8th
Axial stress of 2m distance(MPa)	107.60	165.71	168.11
Axial stress of 4m distance(MPa)	41.16	42.71	37.32

Figure 6.4-12 Axial stress of rock dowel #47 developing with excavation stages

(Two-points anchor bar stress meter)



Excavation stage	6th	7th	8th
Axial stress of 2m distance(MPa)	39.49	36.61	33.10
Axial stress of 4m distance(MPa)	40.34	52.14	47.14

Figure 6.4-11 Axial stress of rock dowel #49 developing with excavation stages

(Two-points anchor bar stress meter)

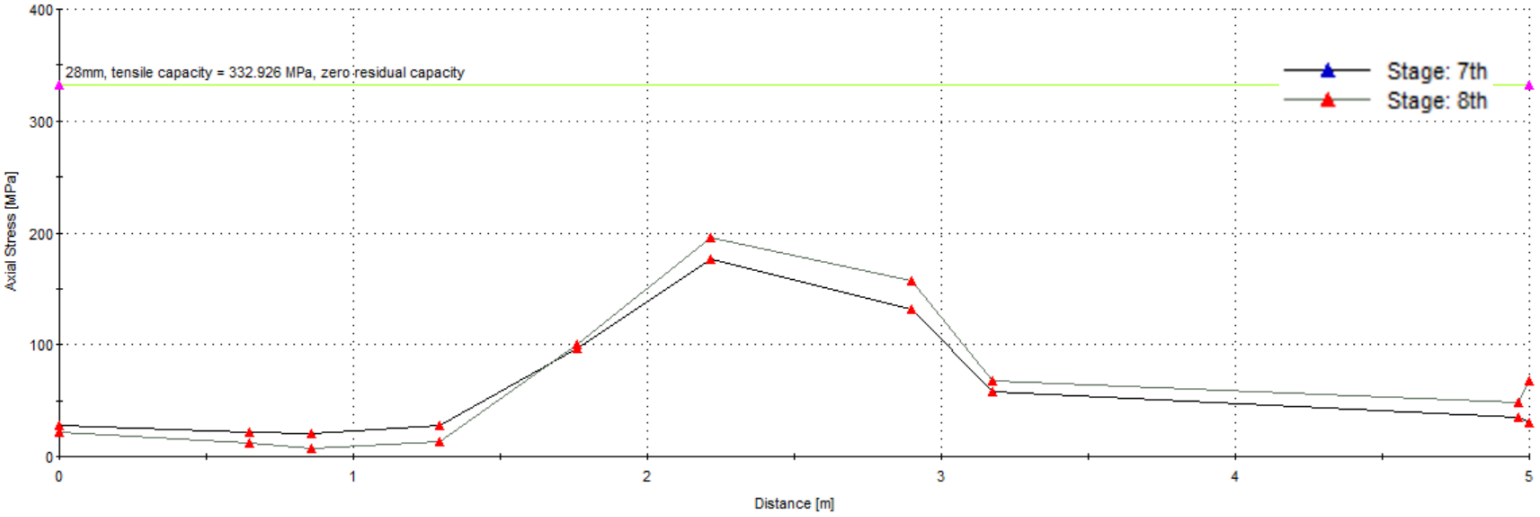


Figure 6.4-13 Axial stress of rock dowel #58 developing with excavation stages

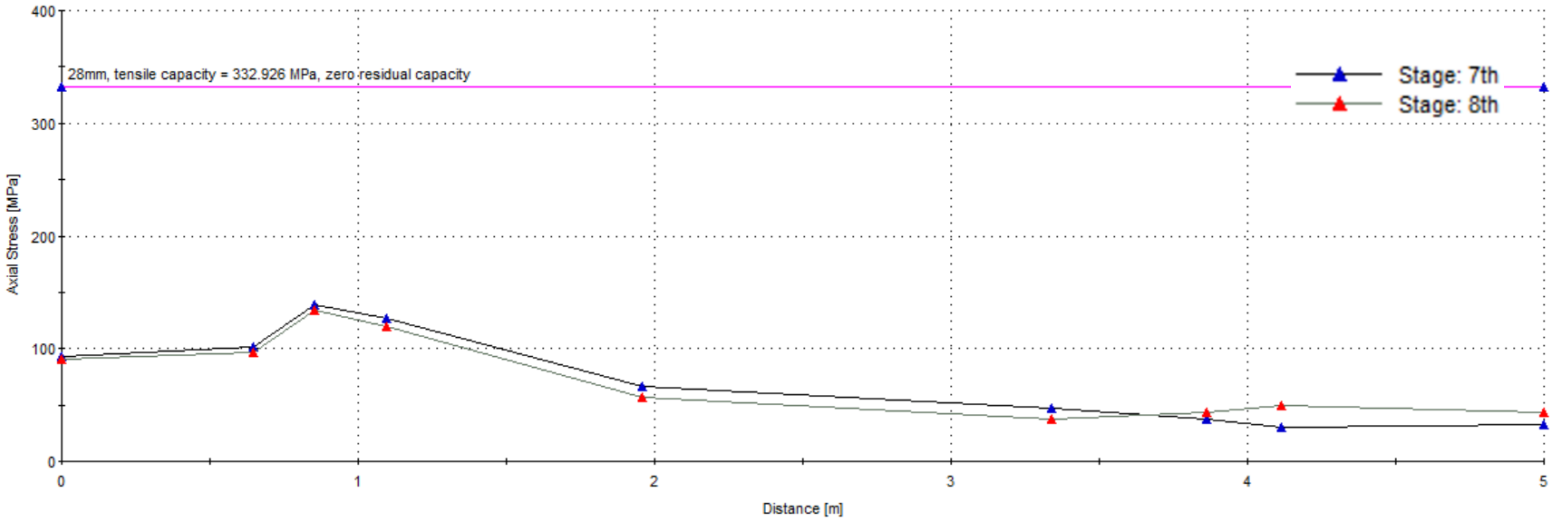


Figure 6.4-14 Axial stress of rock dowel #59 developing with excavation stages

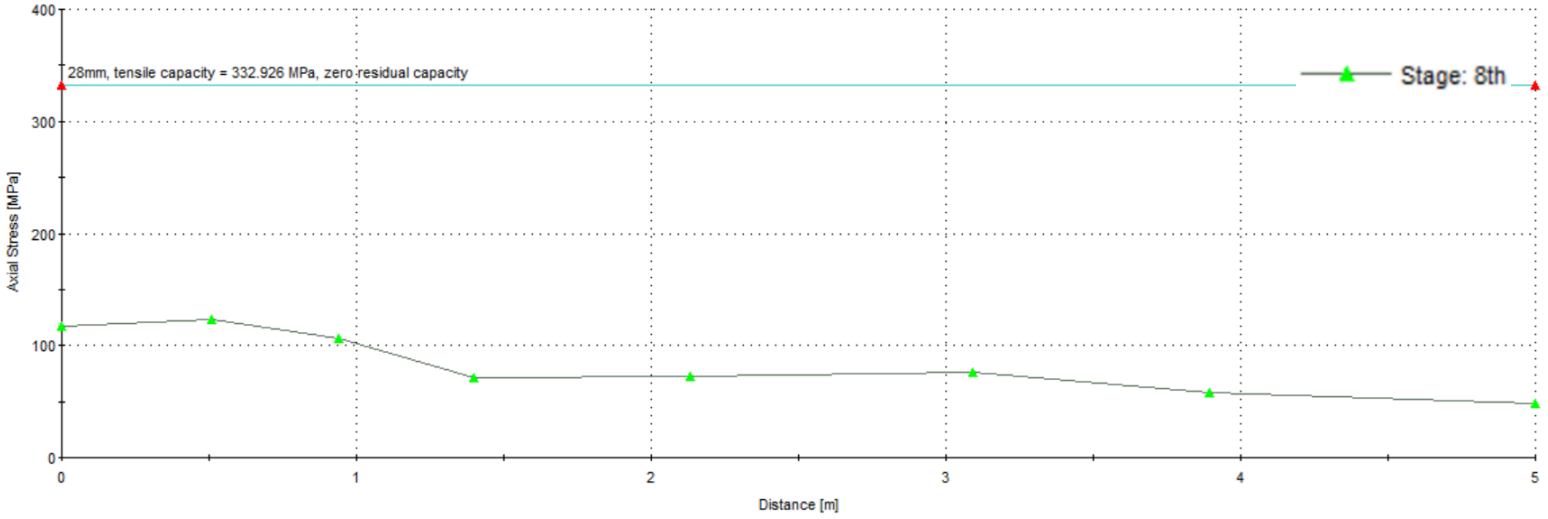


Figure 6.4-15 Axial stress of rock dowel #64 developing with excavation stages

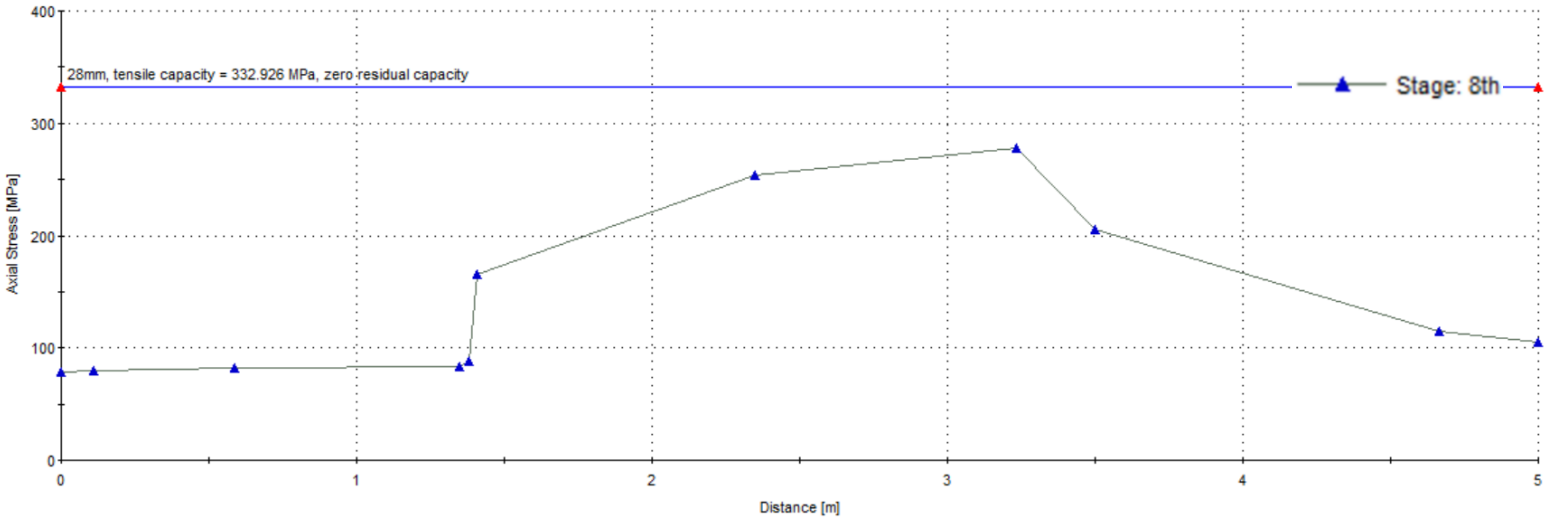


Figure 6.4-16 Axial stress of rock dowel #65 developing with excavation stages

6.5 Rock class III (Basic design supporting scheme)

According to bid documents, annexes, section VII, annex 5, 07 powerhouse drawings, supports of the powerhouse and transformer caverns is as below and shown in Figure 6.5-1.

(1)Powerhouse cavern

Roof: rock dowels, φ32mm, spacing 1.5m, L=8.0m.

Side wall: rock dowels, φ32mm, spacing 1.5m, L=12.0m.

(2)Transformer cavern

Roof: rock dowels, φ32mm, spacing 1.5m, L=8.0m.

Side wall: rock dowels, φ32mm, spacing 1.5m, L=12.0m.

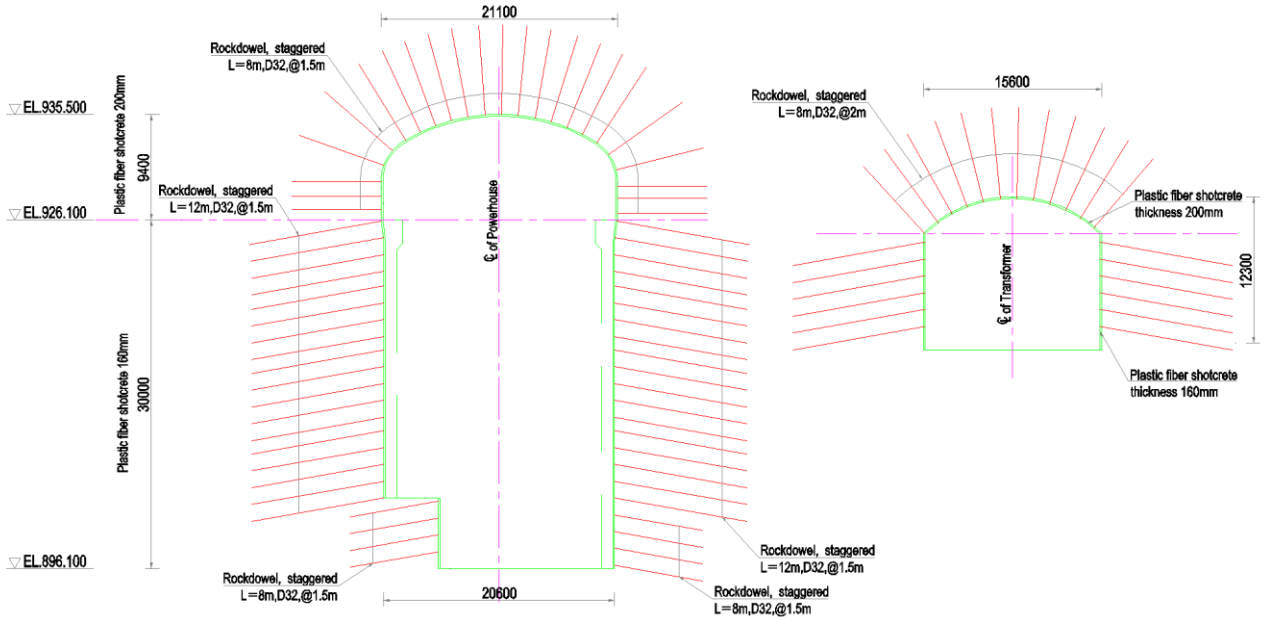


Figure 6.5-1 Basic design supports of powerhouse and transformer caverns (rock type III)

Calculation result is as shown in Figures 6.5-2~6.5-4.

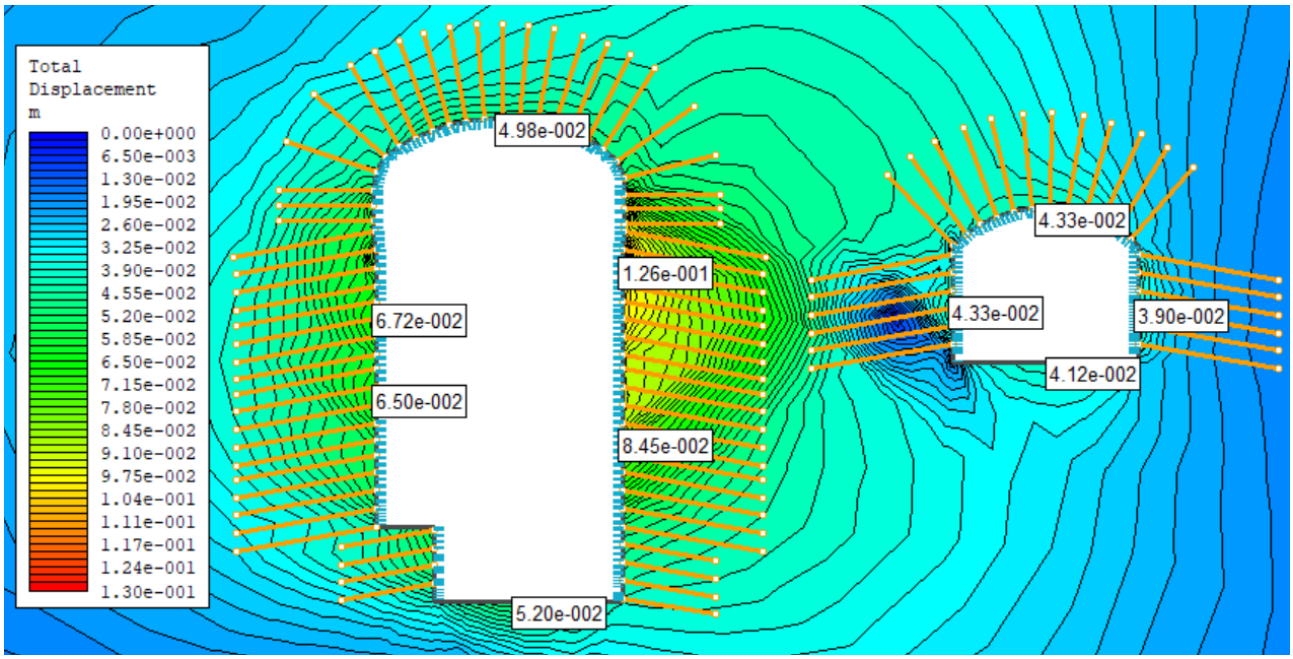


Figure 6.5-2 Total displacement after support installation (Rock Class III)

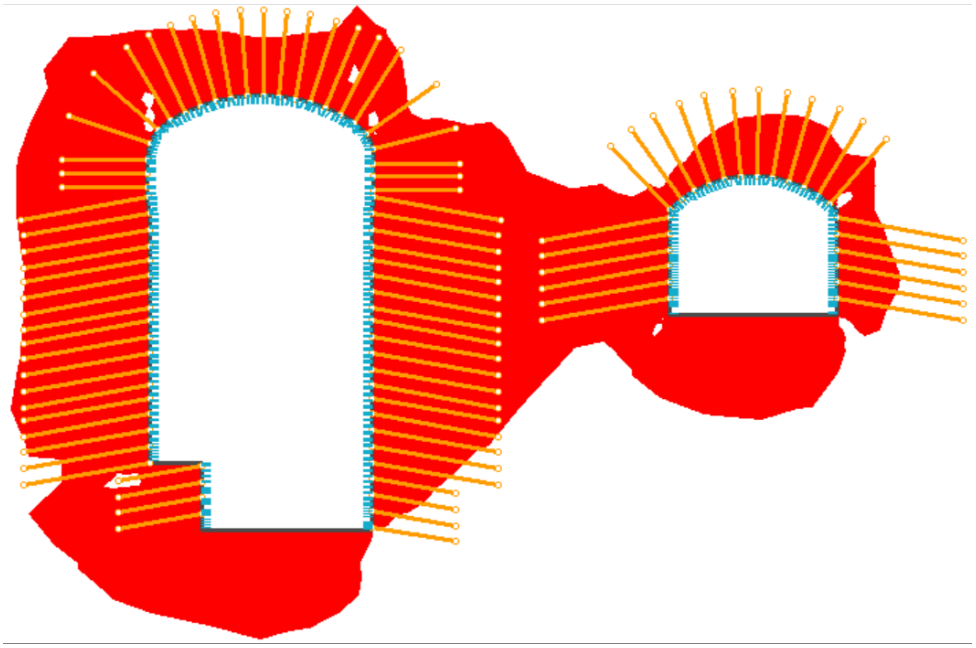


Figure 6.5-3 Plastic zone after support installation (Rock Class III)

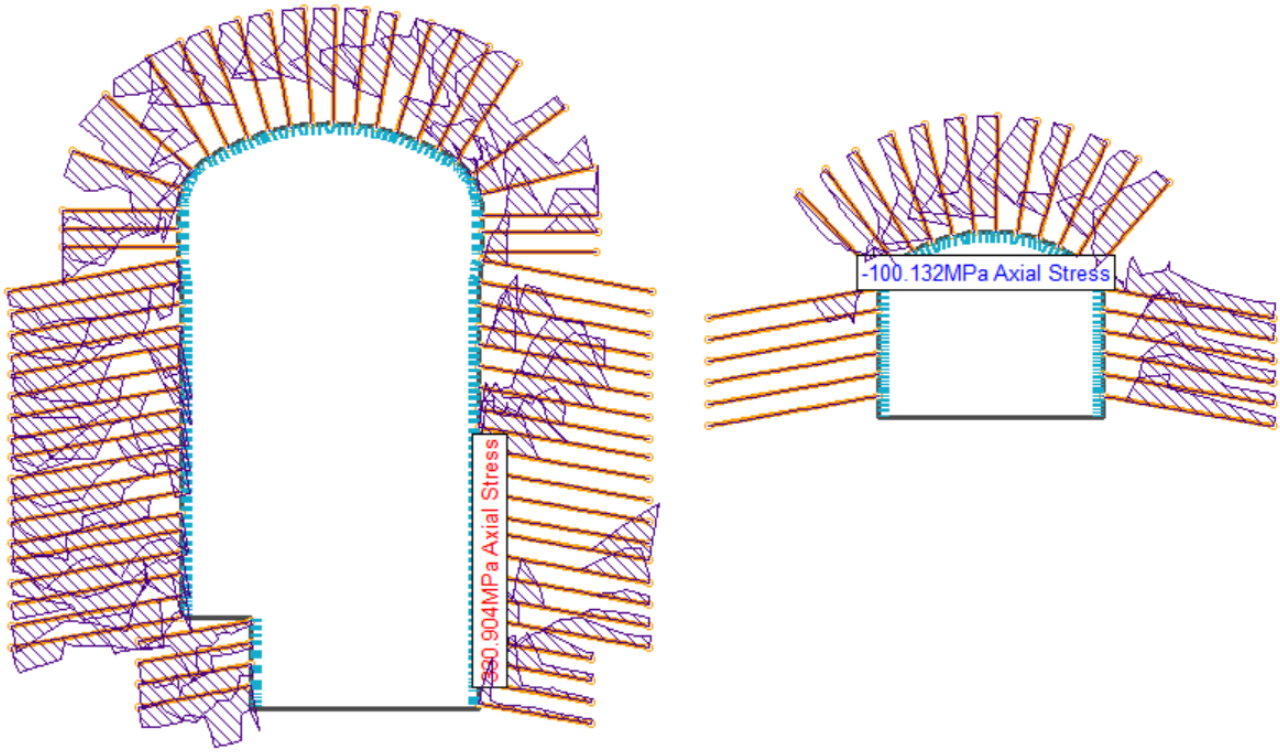


Figure 6.5-4 Axial stress of rock dowels after support installation (Rock Class III)

6.6 Rock class III (Prop supports)

The proposed supports in this stage are almost the same as that proposed in basic design stage except supports in the central portion of the cavern crowns, where, rock bolts are proposed to supersede the rock dowels and vertically arranged.

The proposed supports in this stage are as shown in Figure 6.6-1.

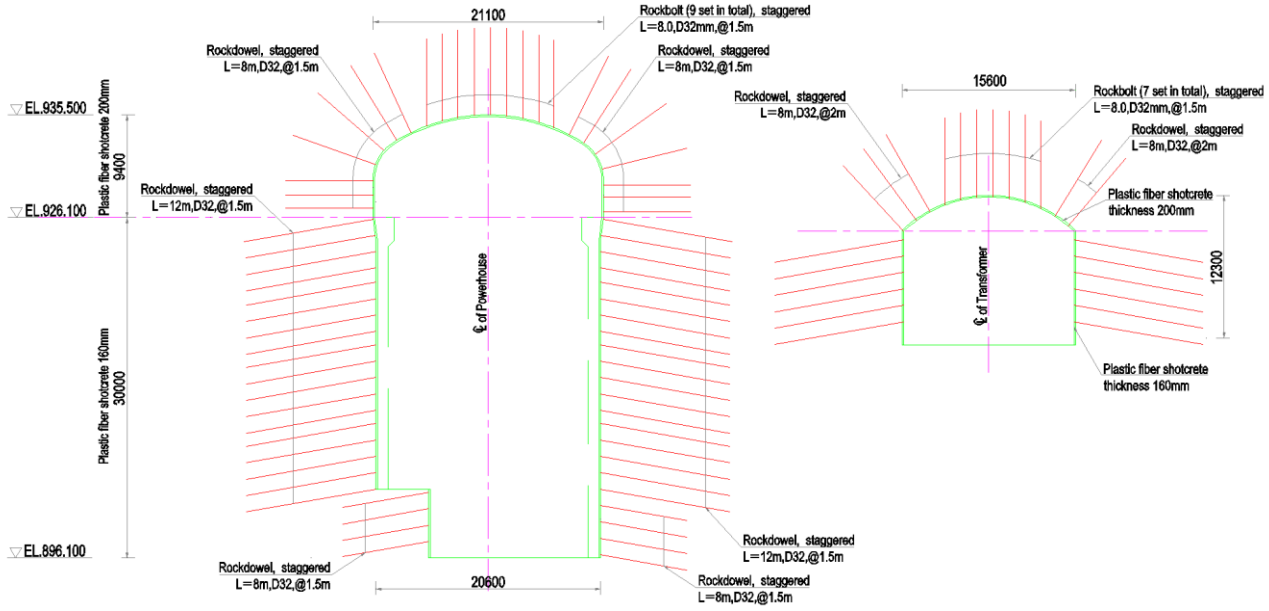


Figure 6.6-1 Prop supports of powerhouse and transformer caverns (rock type III)

Calculation result is as shown in Figures 6.6-2~6.6-4.

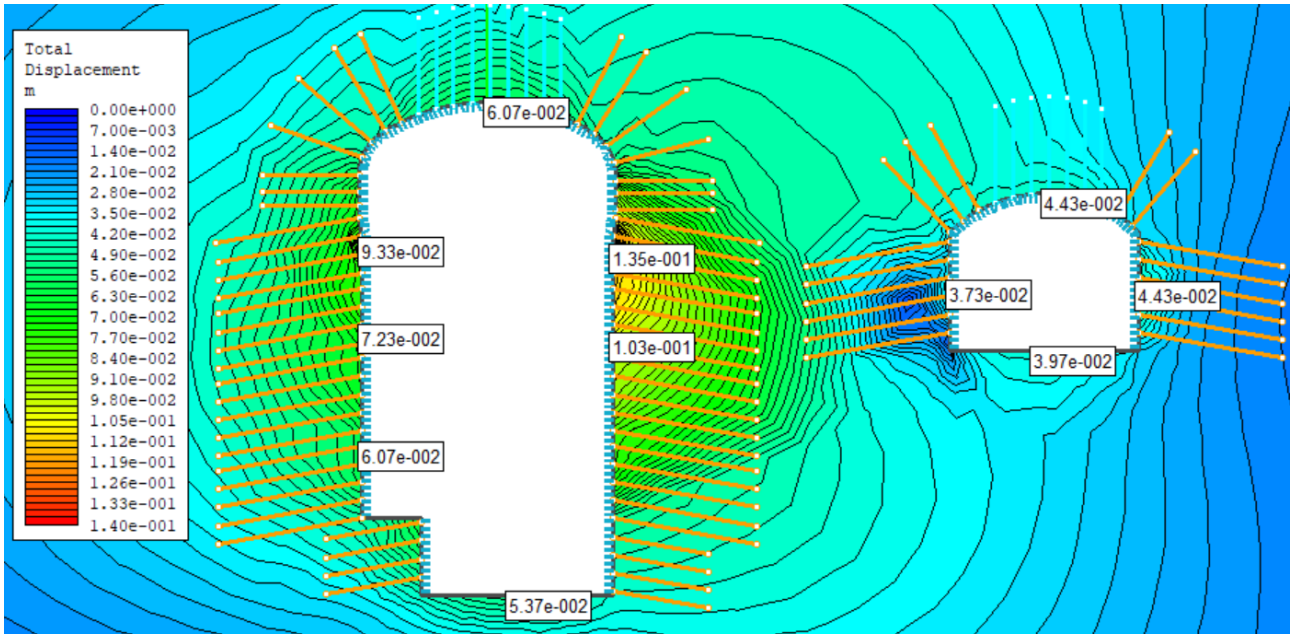


Figure 6.6-2 Total displacement after support installation (Rock Class III)

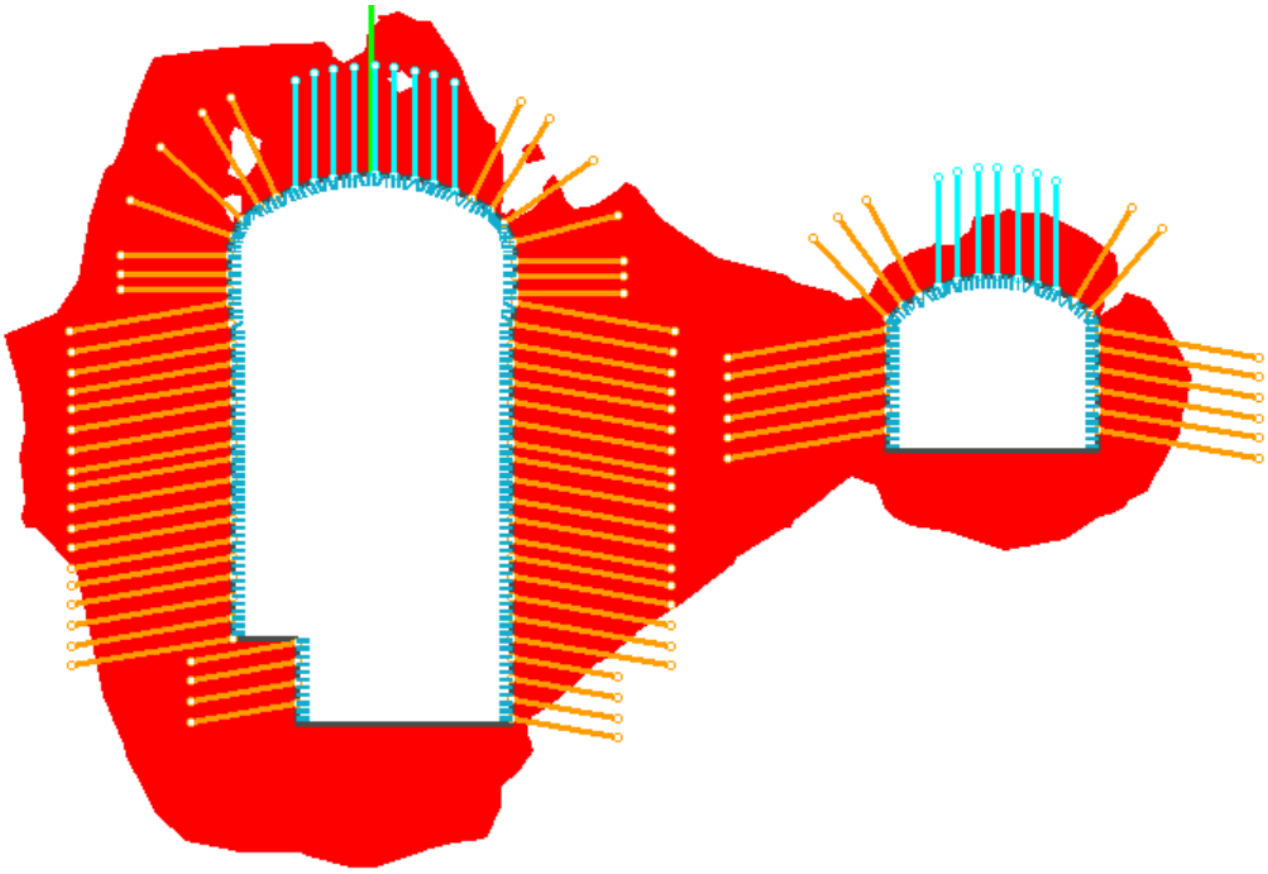


Figure 6.6-3 Plastic zone after support installation (Rock Class III)

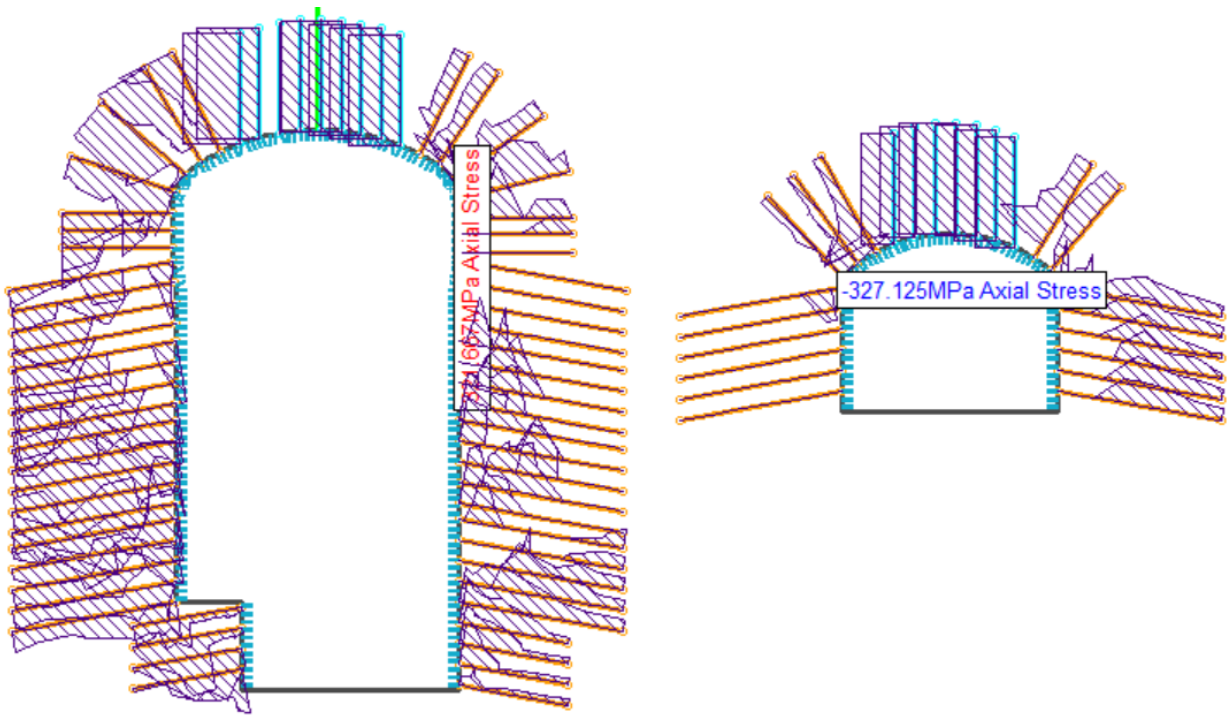


Figure 6.6-4 Axial stress of rock dowels/rock bolts after support installation (Rock Class III)

6.7 Displacement of multi-point extensometer of powerhouse cavern crown (Rock class III,prop supports)

According to FEM calculation result, displacement of monitoring points (multi-point extensometer) at each excavation stage are shown in figure 6.7-1~8. Displacement histories of each monitoring points are shown in figure 6.7-9~28.

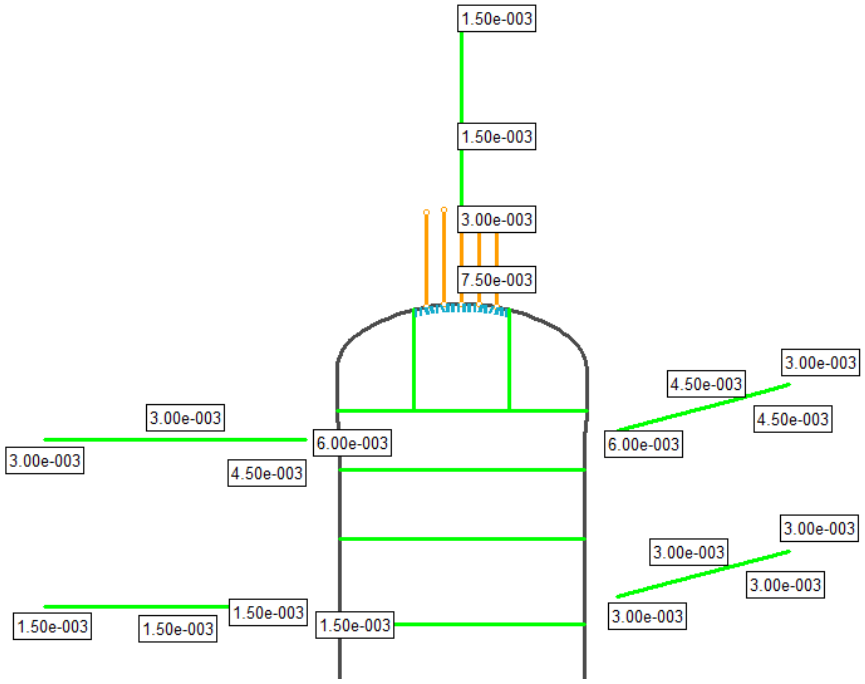


Figure 6.7-1 Displacement of monitoring points at 1st stage (Unit: m)

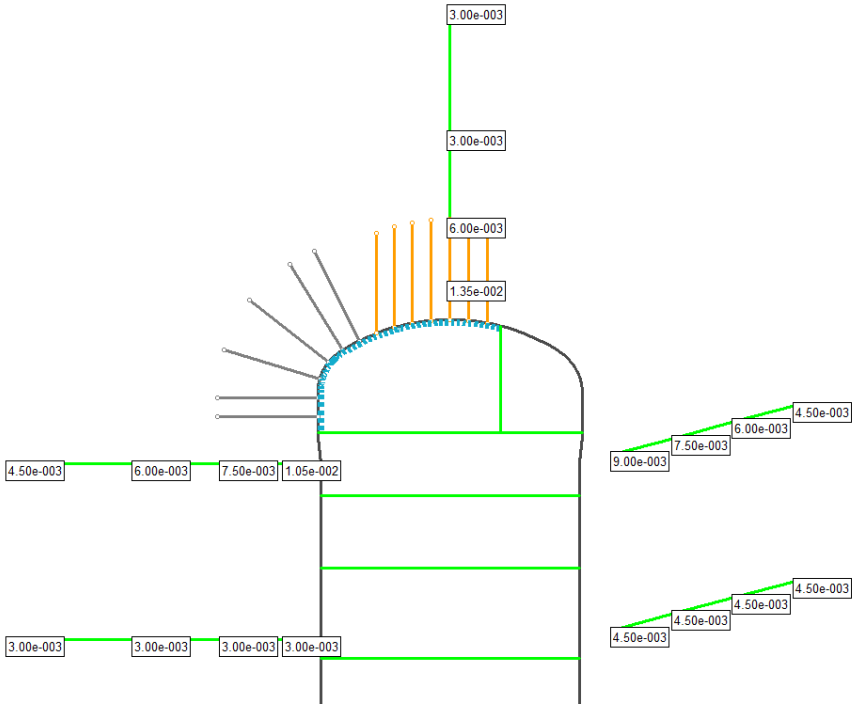


Figure 6.7-2 Displacement of monitoring points at 2nd stage (Unit: m)

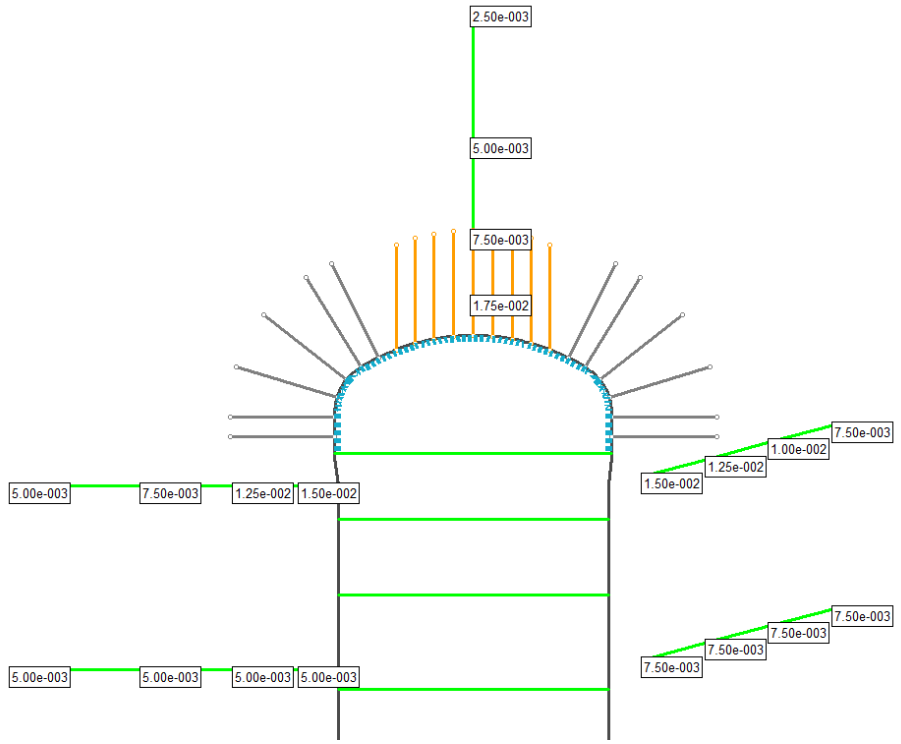


Figure 6.7-3 Displacement of monitoring points at 3rd stage (Unit: m)

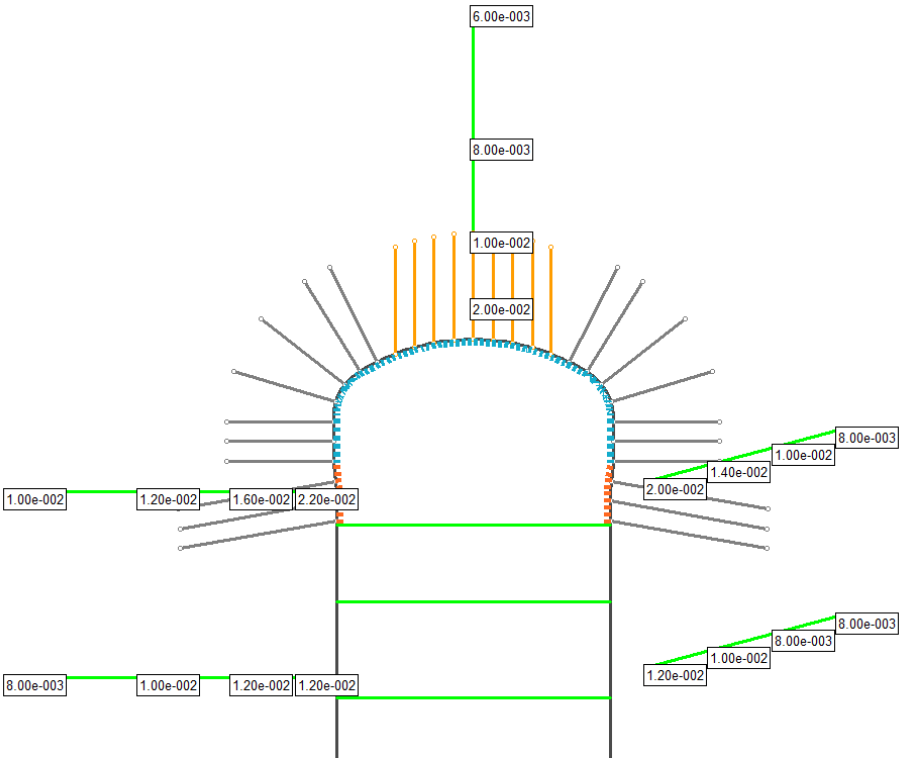


Figure 6.7-4 Displacement of monitoring points at 4th stage (Unit: m)

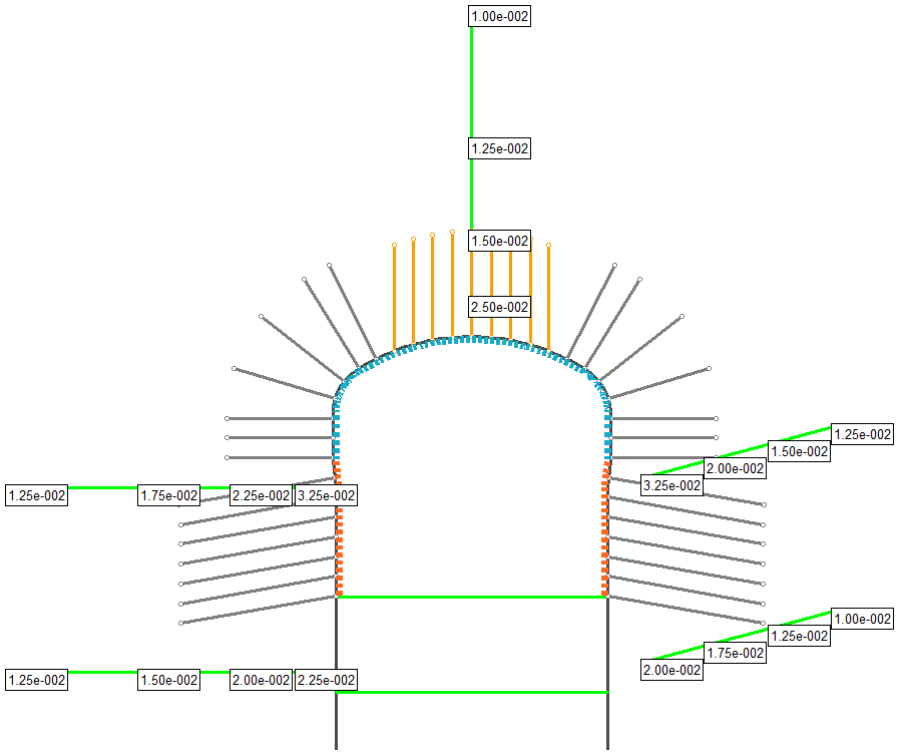


Figure 6.7-5 Displacement of monitoring points at 5th stage (Unit: m)

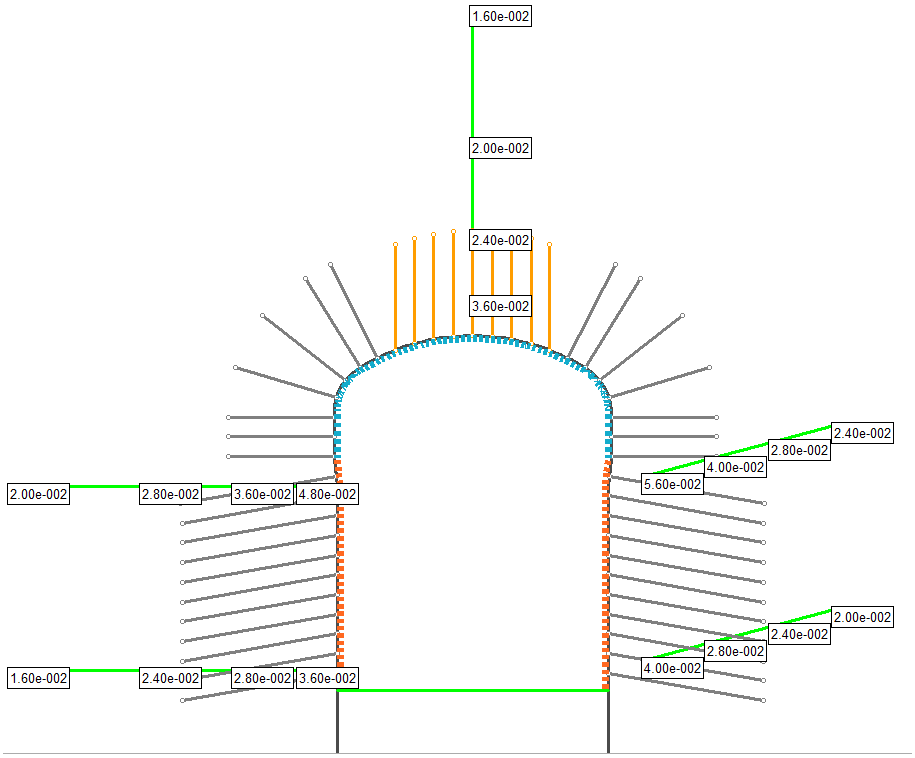


Figure 6.7-6 Displacement of monitoring points at 6th stage (Unit: m)

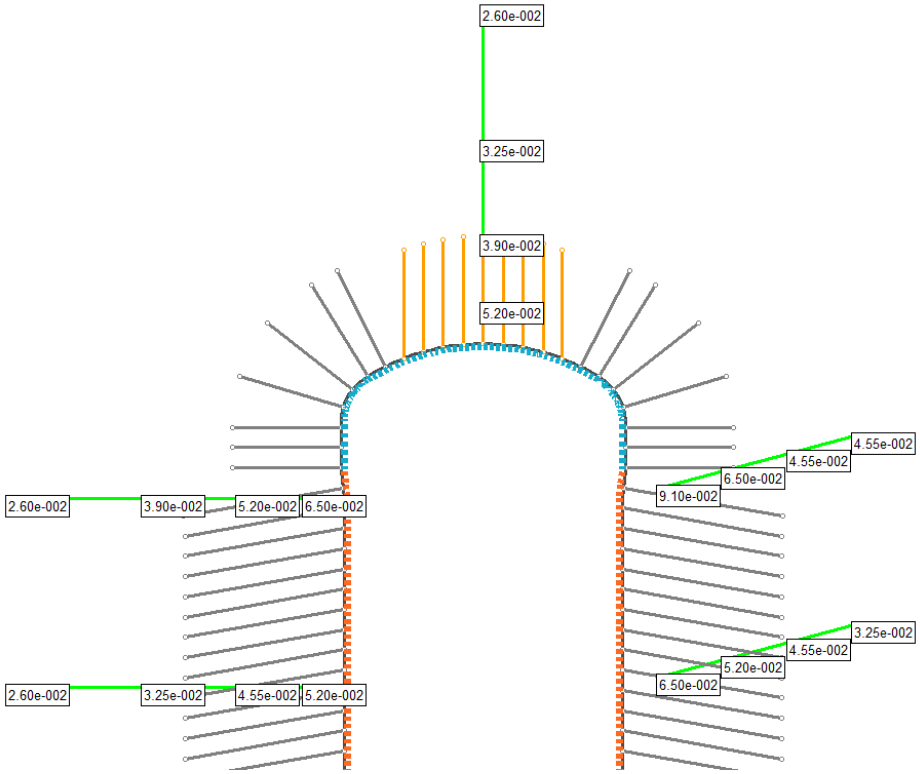


Figure 6.7-7 Displacement of monitoring points at 7th stage (Unit: m)

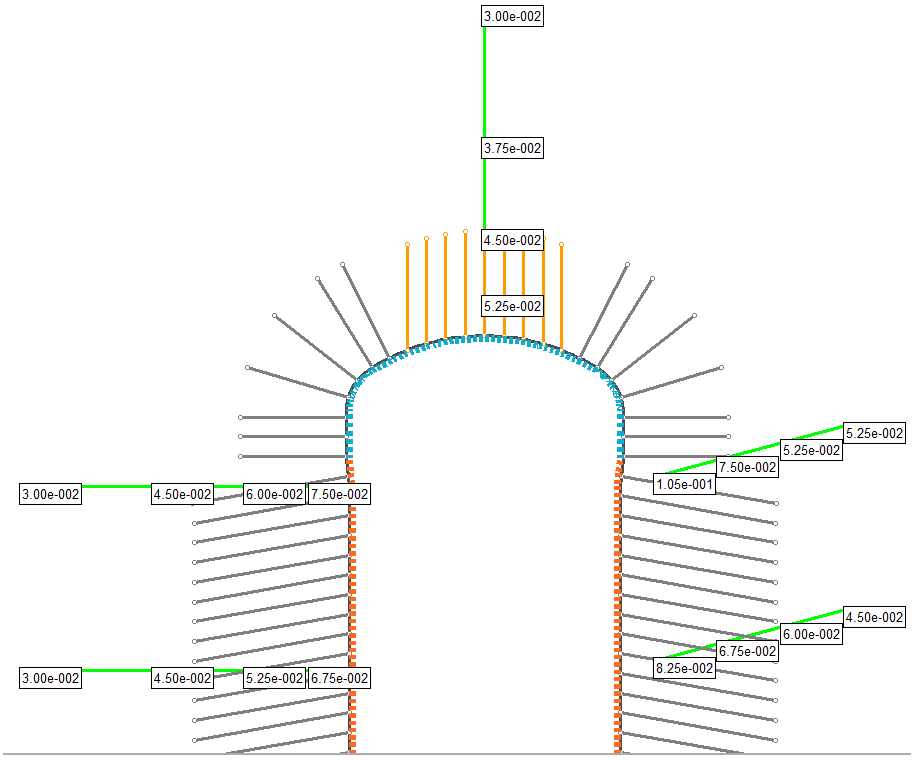


Figure 6.7-8 Displacement of monitoring points at 8th stage (Unit: m)

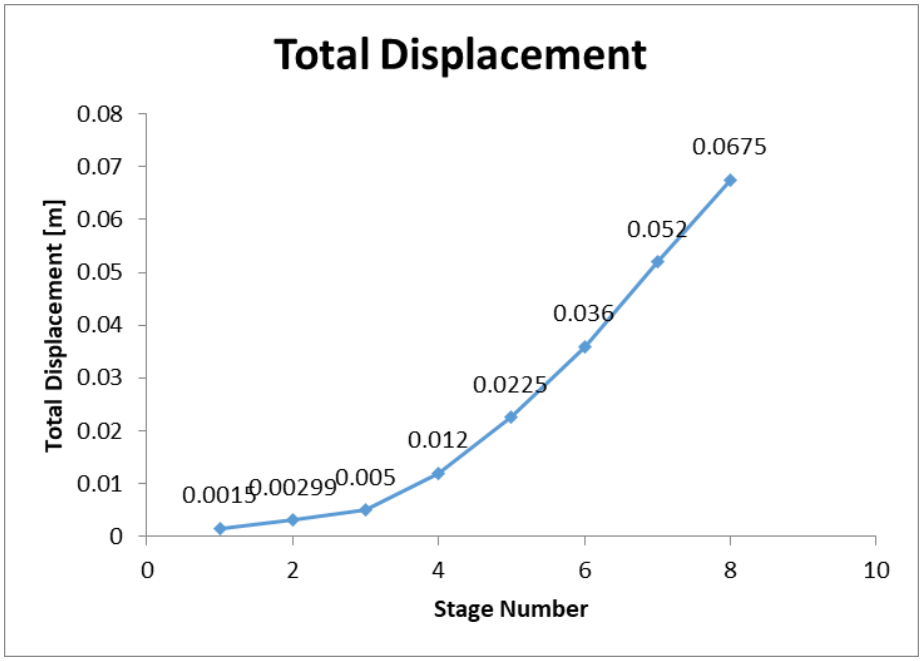


Figure 6.7-9 Displacement history of ME4-1.1 monitoring points

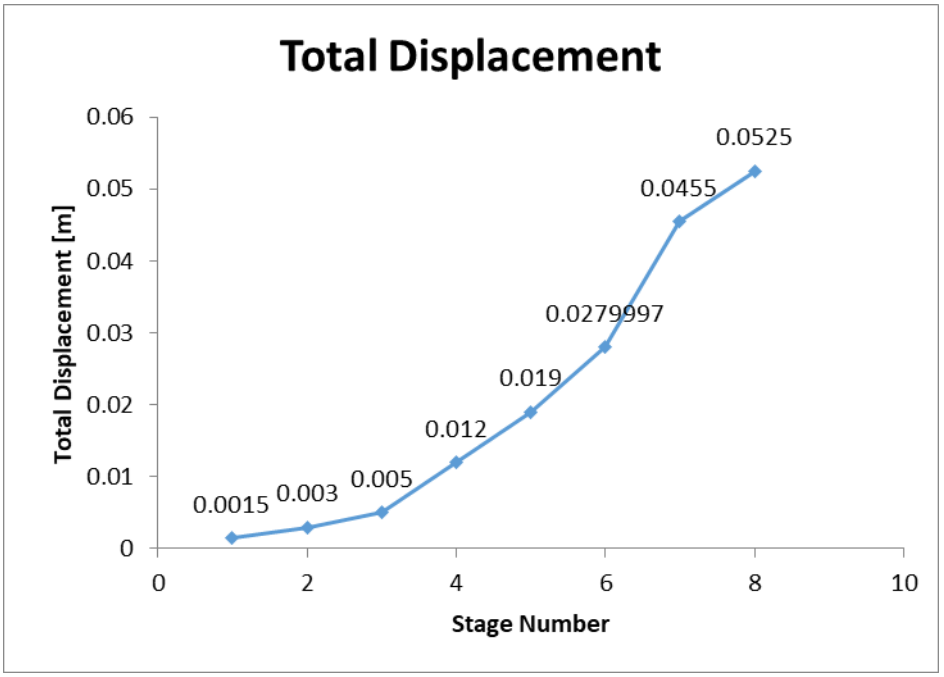


Figure 6.7-10 Displacement history of ME4-1.2 monitoring points

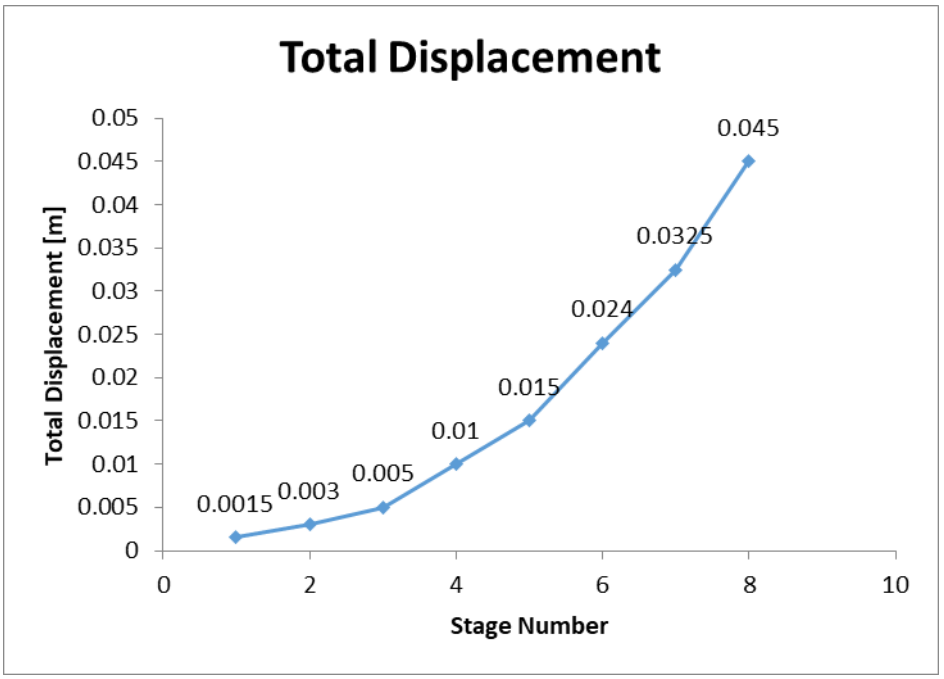


Figure 6.7-11 Displacement history of ME4-1.3 monitoring points

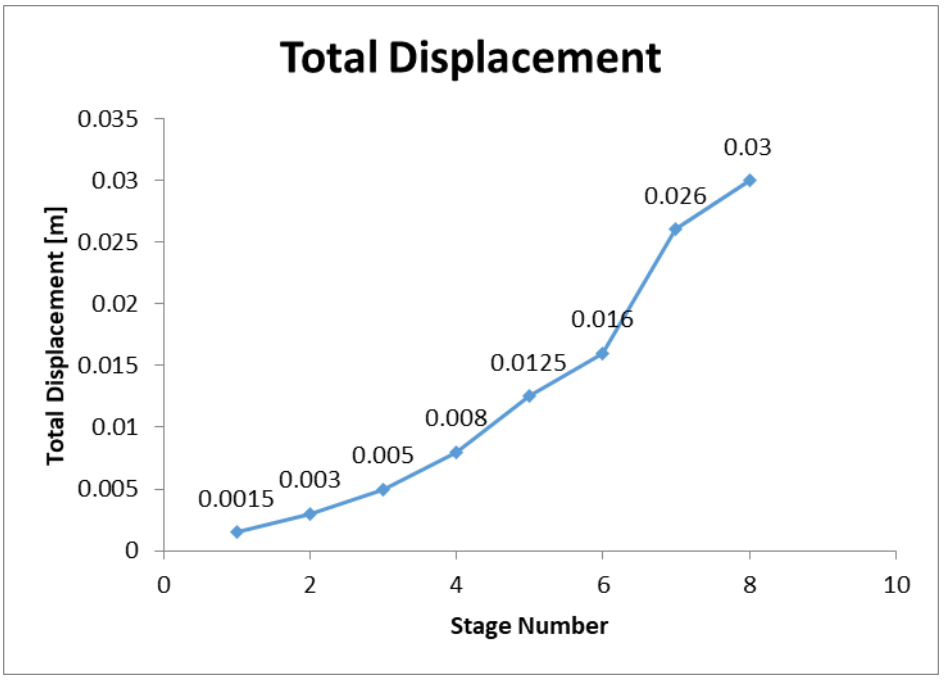


Figure 6.7-12 Displacement history of ME4-1.4 monitoring points

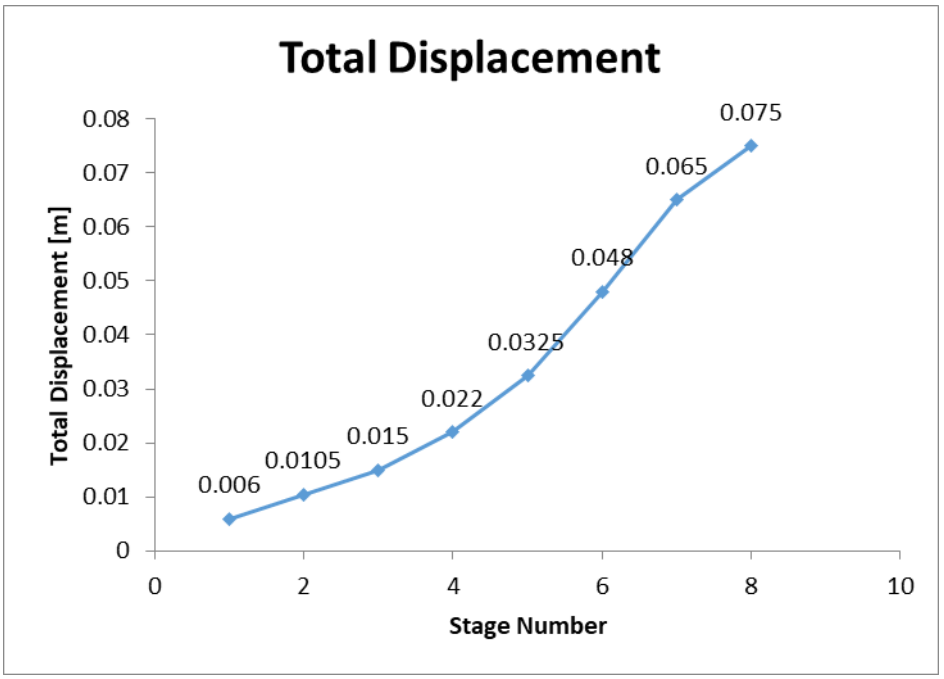


Figure 6.7-13 Displacement history of ME4-2.1 monitoring points

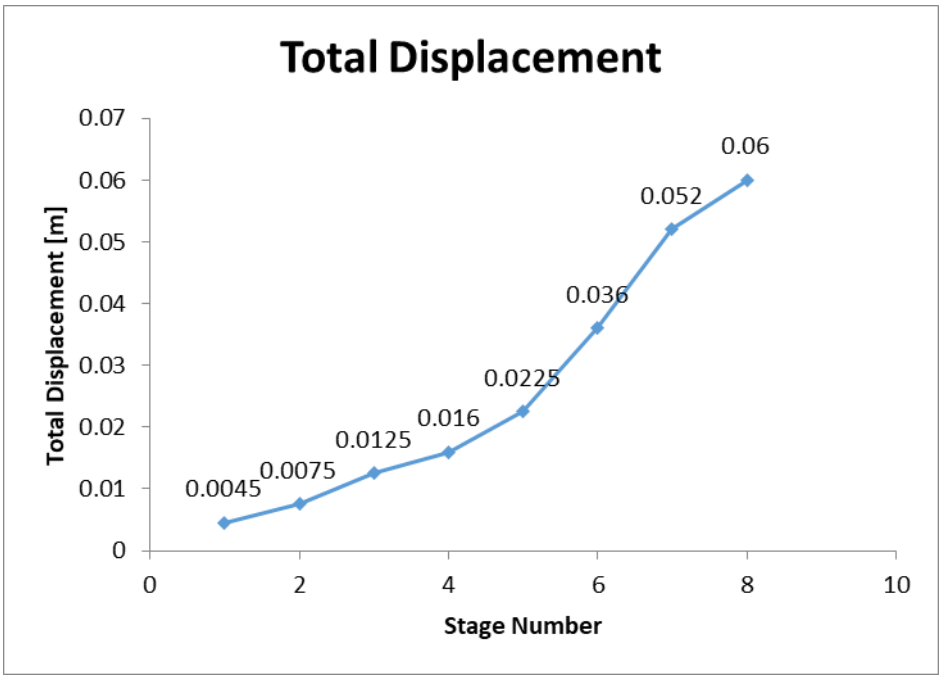


Figure 6.7-14 Displacement history of ME4-2.2 monitoring points

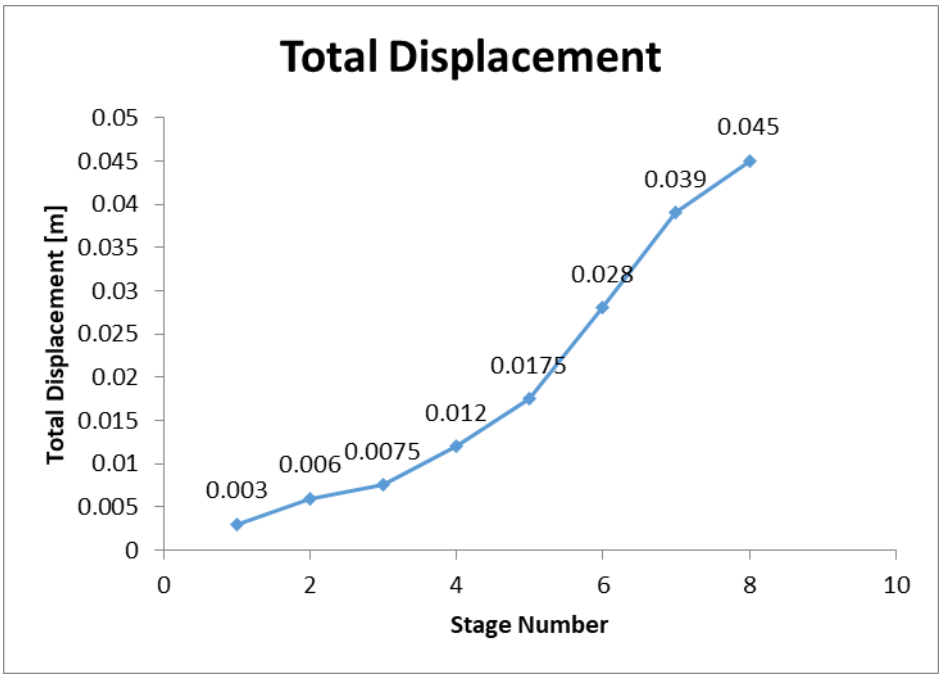


Figure 6.7-15 Displacement history of ME4-2.3 monitoring points

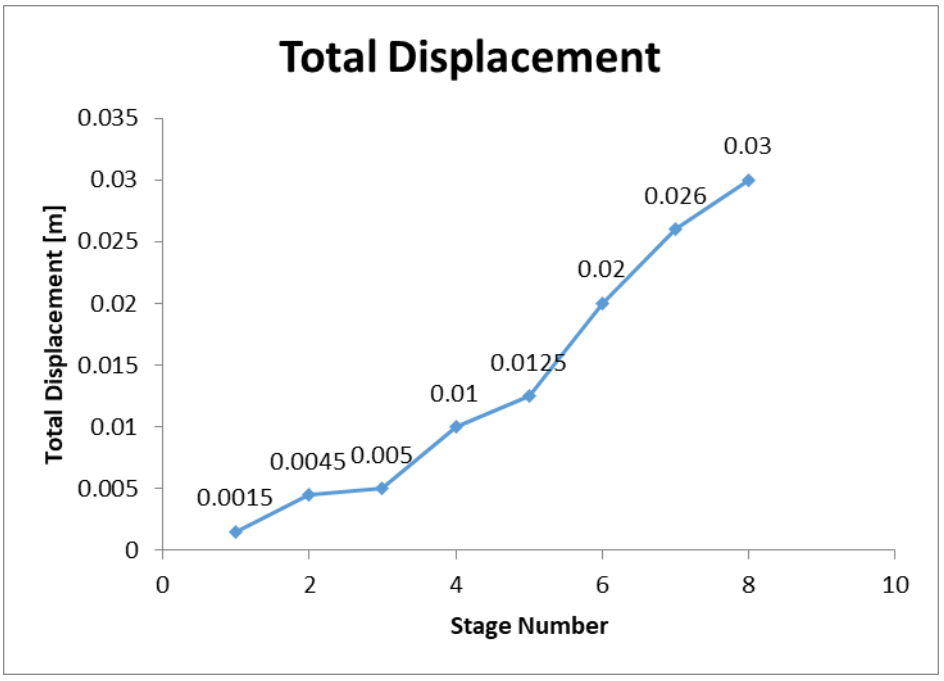


Figure 6.7-16 Displacement history of ME4-2.4 monitoring points

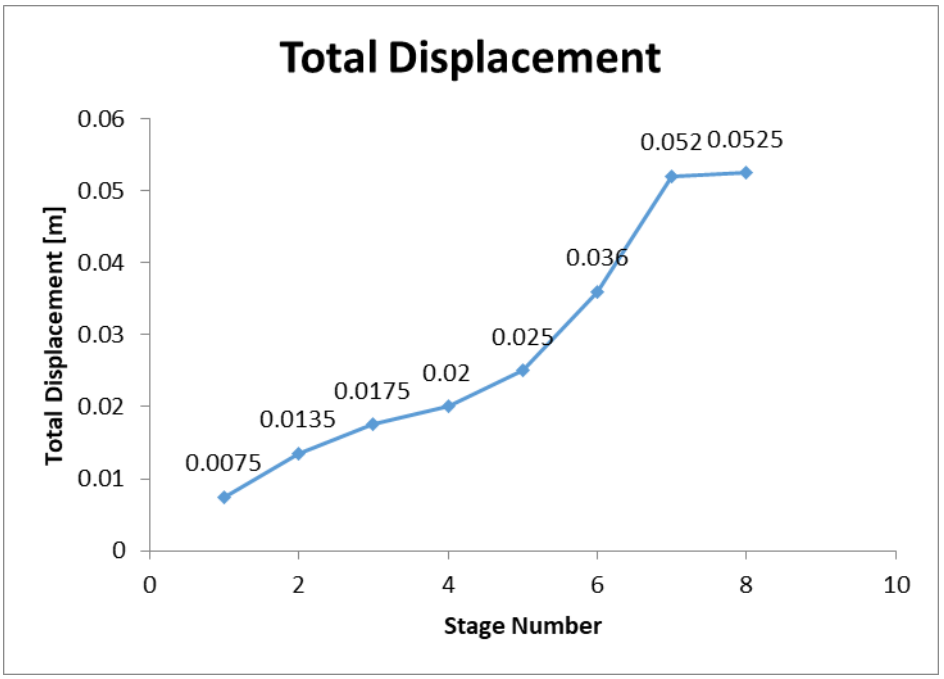


Figure 6.7-17 Displacement history of ME4-3.1 monitoring points

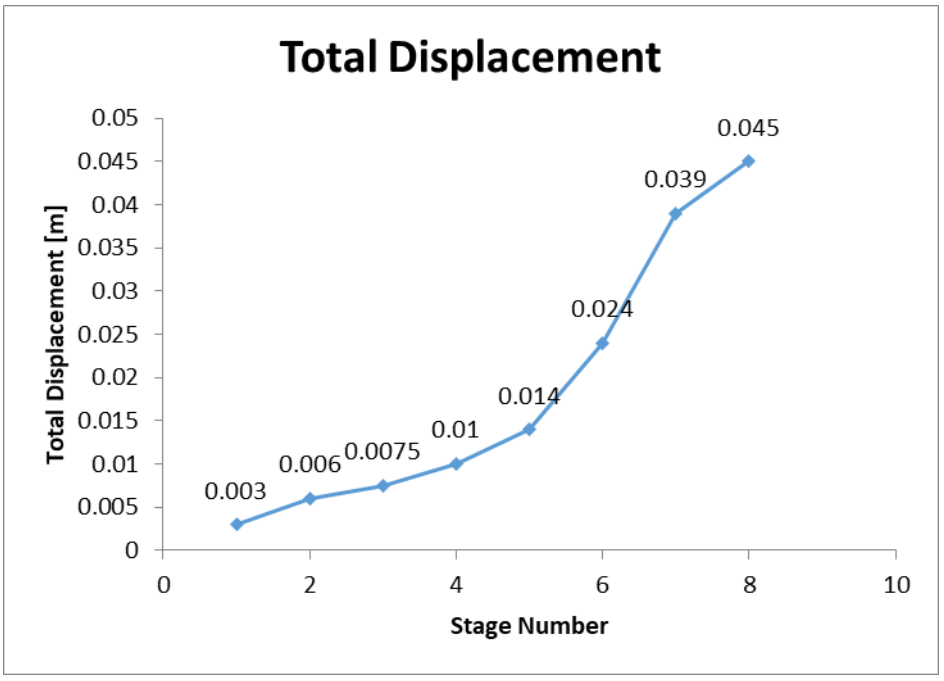


Figure 6.7-18 Displacement history of ME4-3.2 monitoring points

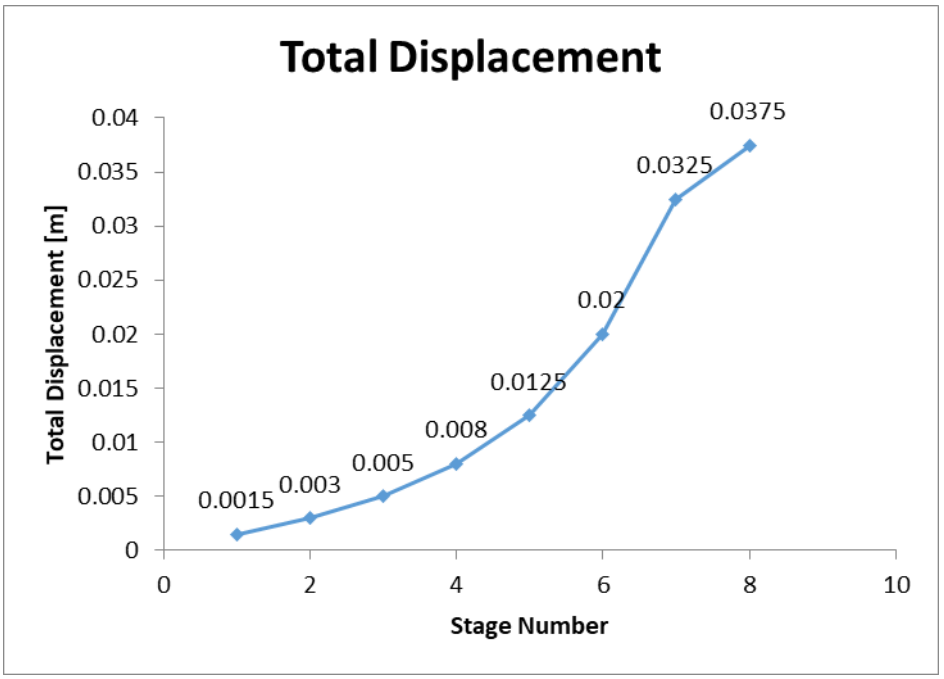


Figure 6.7-19 Displacement history of ME4-3.3 monitoring points

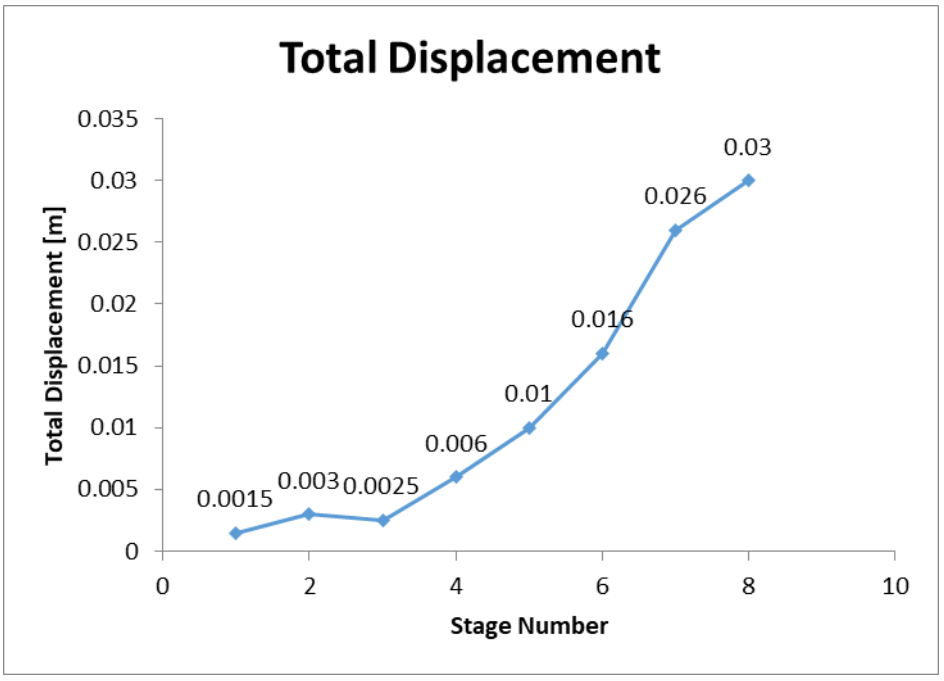


Figure 6.7-20 Displacement history of ME4-3.4 monitoring points

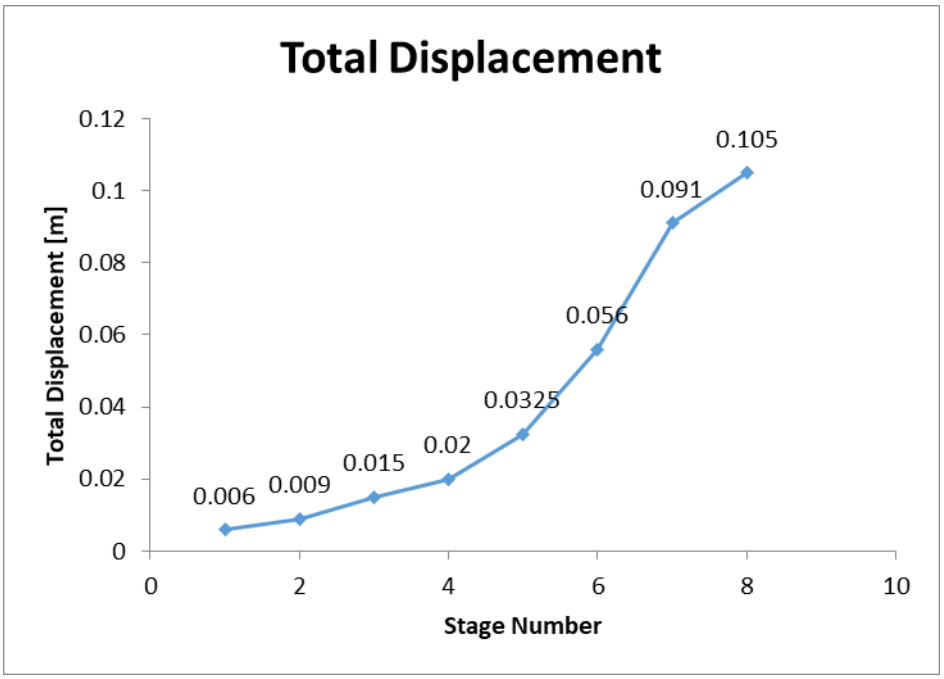


Figure 6.7-21 Displacement history of ME4-4.1 monitoring points

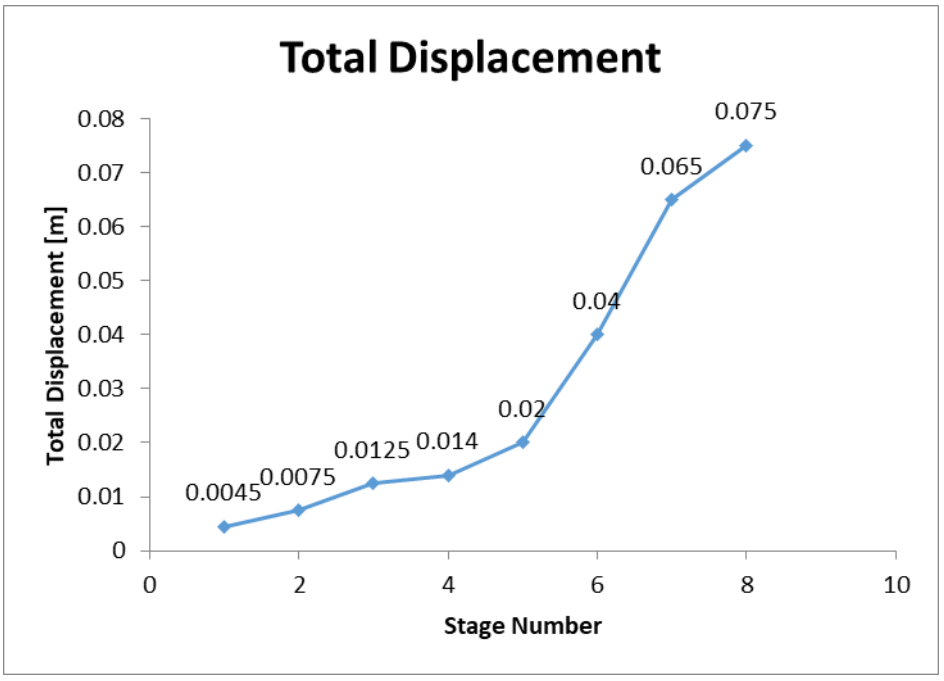


Figure 6.7-22 Displacement history of ME4-4.2 monitoring points

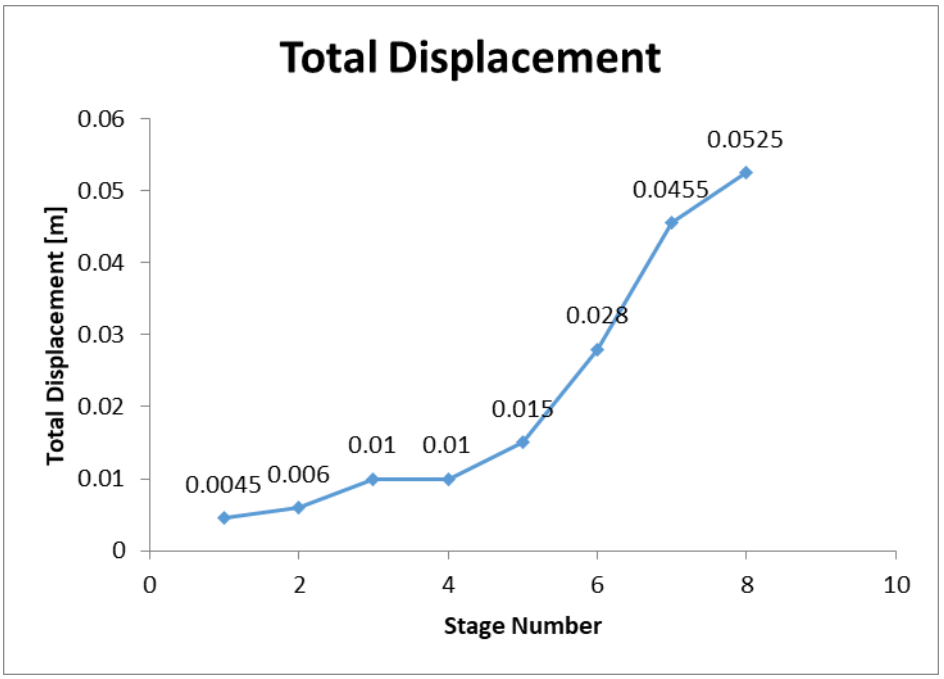


Figure 6.7-23 Displacement history of ME4-4.3 monitoring points

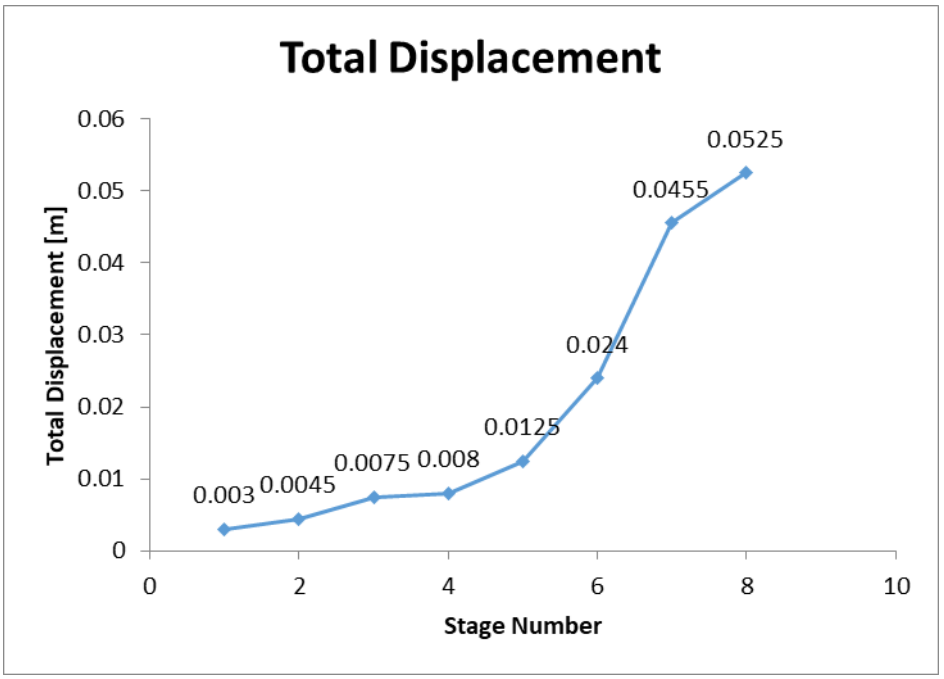


Figure 6.7-24 Displacement history of ME4-4.4 monitoring points

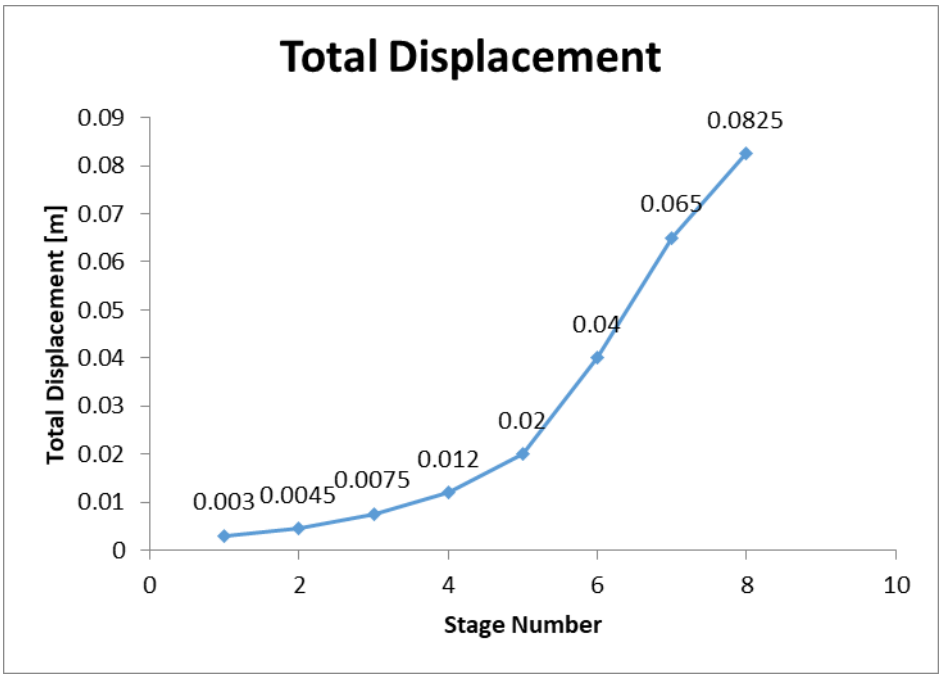


Figure 6.7-25 Displacement history of ME4-5.1 monitoring points

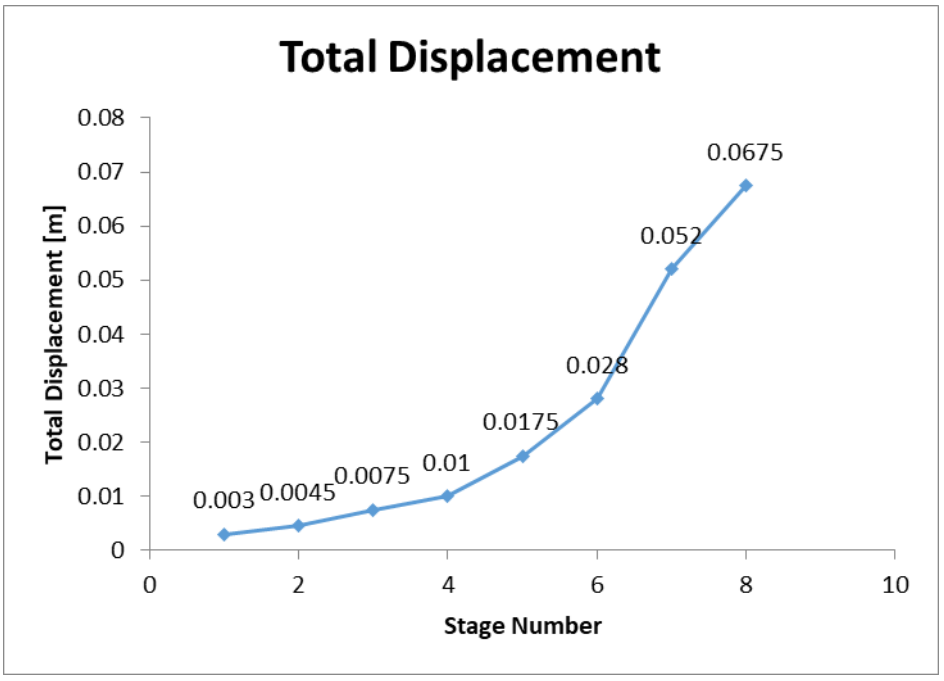


Figure 6.7-26 Displacement history of ME4-5.2monitoring points

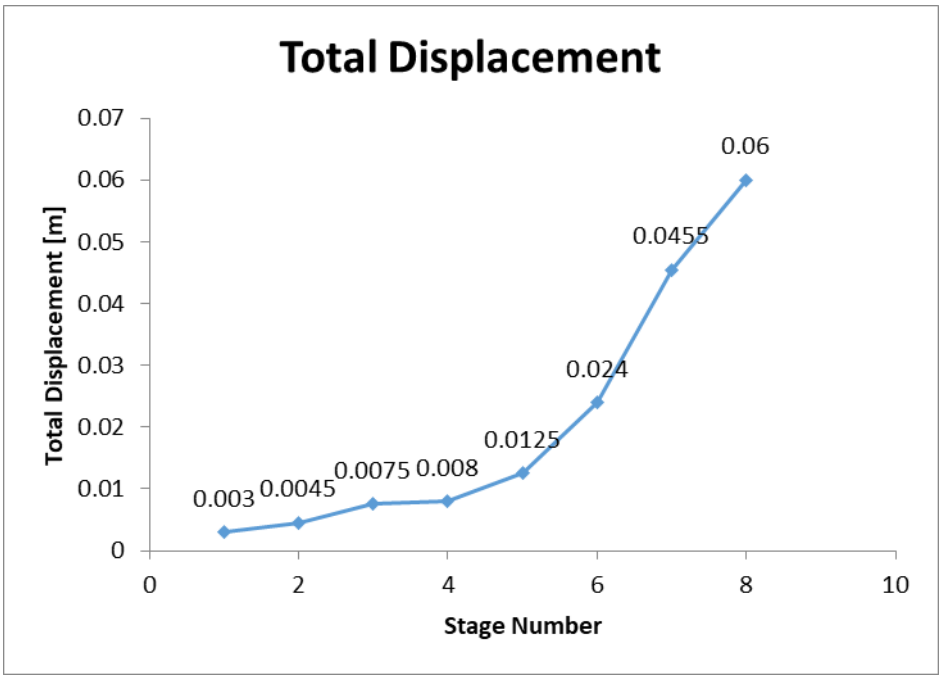


Figure 6.7-27 Displacement history of ME4-5.3monitoring points

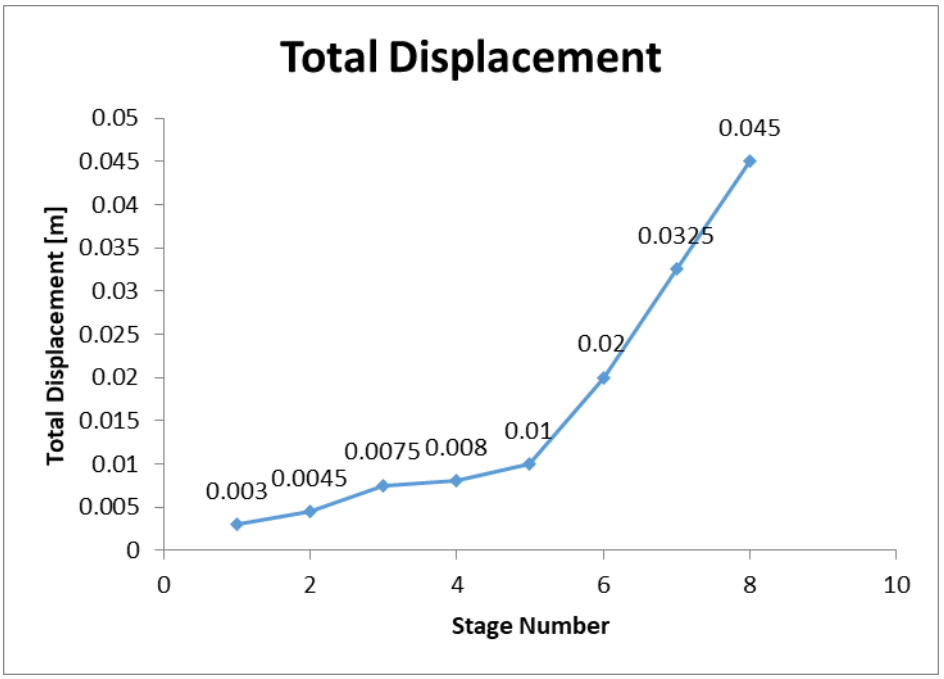


Figure 6.7-28 Displacement history of ME4-5.4monitoring points

7 Wedge Analysis based Predictive Parameters

7.1 Discontinuity Input Data

According to the geological logging of the ventilation tunnel and the main access tunnel of the power house and the geological mapping around the power house, a total of 615 joints are counted. Most joints are closed, a few are slightly open, there is no filling between joints, and a few joints are filled with a small amount of calcium film, rock powder and rock debris.

According to the statistical analysis of Dips software, four groups of joints are mainly developed in the powerhouse area, and the joint characteristics are as follows:

There are 4 groups joints may distribute in the powerhouse surrounding rock, as is shown in Table 7-1~7-2. The stereographic projection of these joints in rock mass in shown in Figure 7-1~7-4. The Mechanical parameter of joints is shown in Table 7-3.

Table 7-1 Description of joints within the powerhouse cavern surrounding rock

NO.	Type	Occurrence（dip direction/dip）	Structural plane
J1	Foliation	355-10/15-30	spacing is 0.1-0.3m, the local spacing is 0.01-0.05m, the persistence > 12m, the surface is straight and rough
J2	joint	135-150/60-75	spacing is 0.5-2m, the persistence 2-8m, the surface is straight and rough
J3	joint	198-208/68-75	spacing is 2-5m, the persistence 2-8m, the surface is straight and rough
J4	joint	20-45/65-72	spacing is 0.5-2m, the persistence 0.5-4m, the surface is straight and rough

Table 7-2 Altitudes of joints adopted in wedge stability analysis

Joint NO.	Dip	Dip Direction	powerhouse Direction(Trend)
J1(foliation)	25°	5°	N93°
J2	70°	145°	
J3	72°	200°	
J4	70°	30°	

Table 7-3 Mechanical parameter of joints within the powerhouse cavern surrounding rock

JONIT TYPE	C	Φ
	（MPa）	（°）
Foliation(J1)	0.20	35.0
lithoclasts cemented (J2~J6)	0.15	28.8

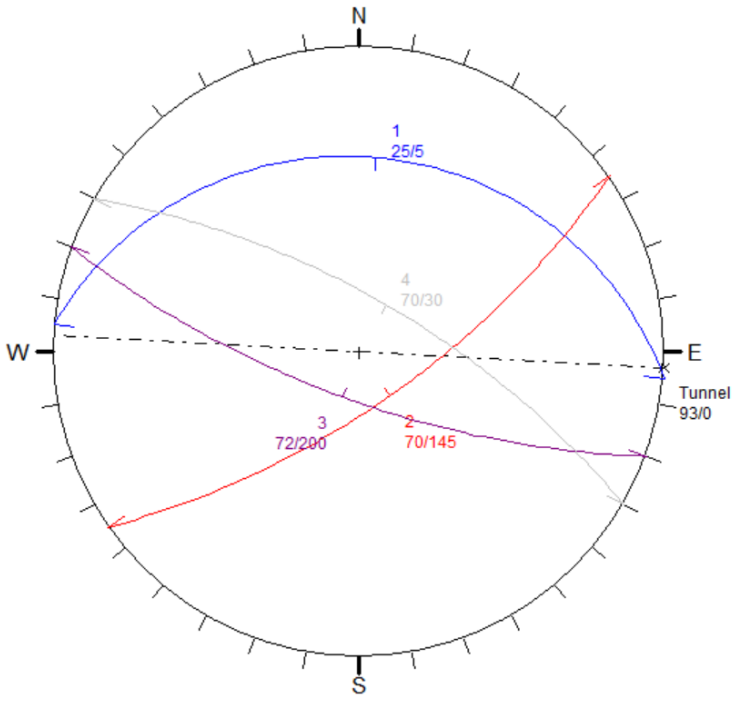


Figure 7-1 Stereographic projection of the joins J1/J2/J3

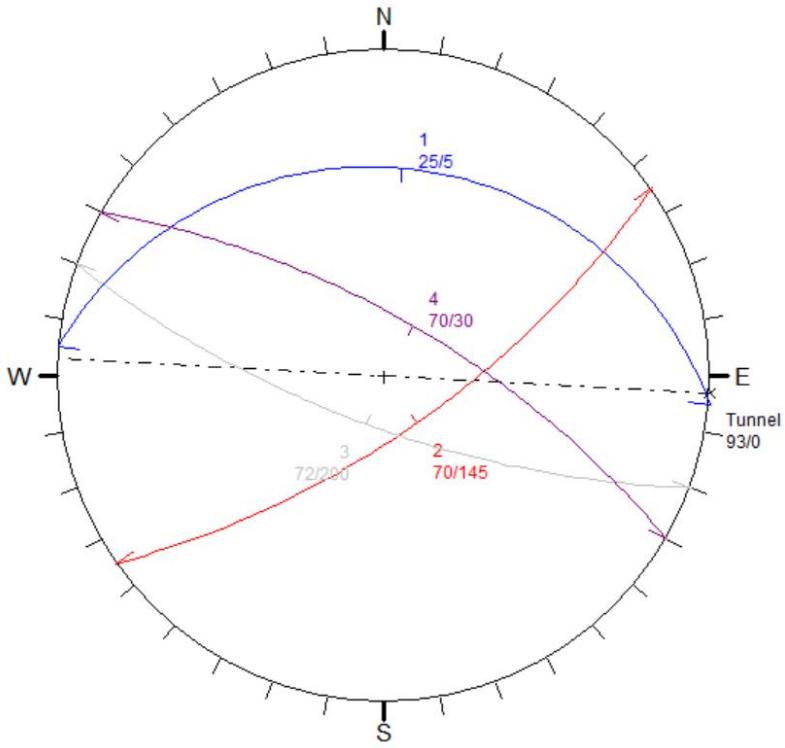


Figure 7-2 Stereographic projection of the joins J1/J2/J4

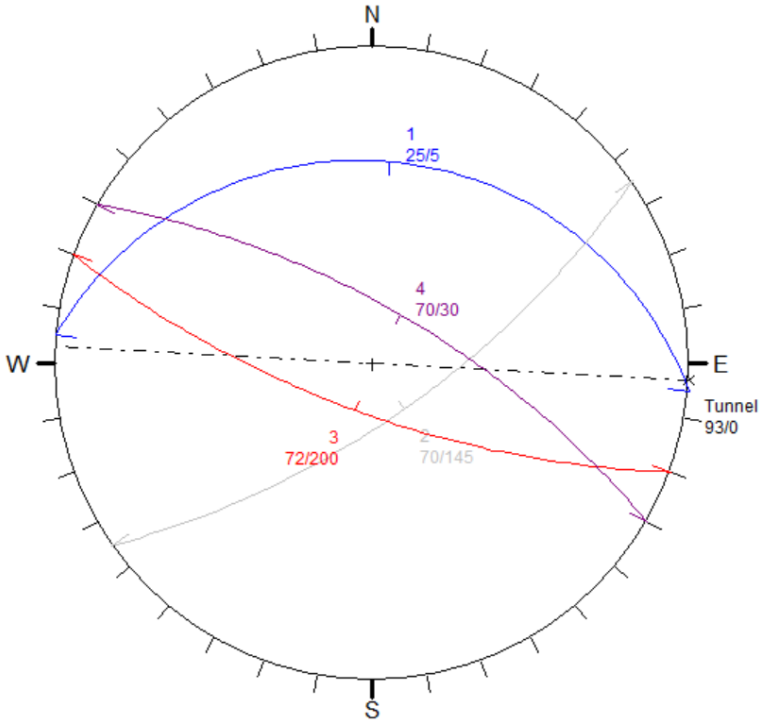


Figure 7-3 Stereographic projection of the joins J1/J3/J4

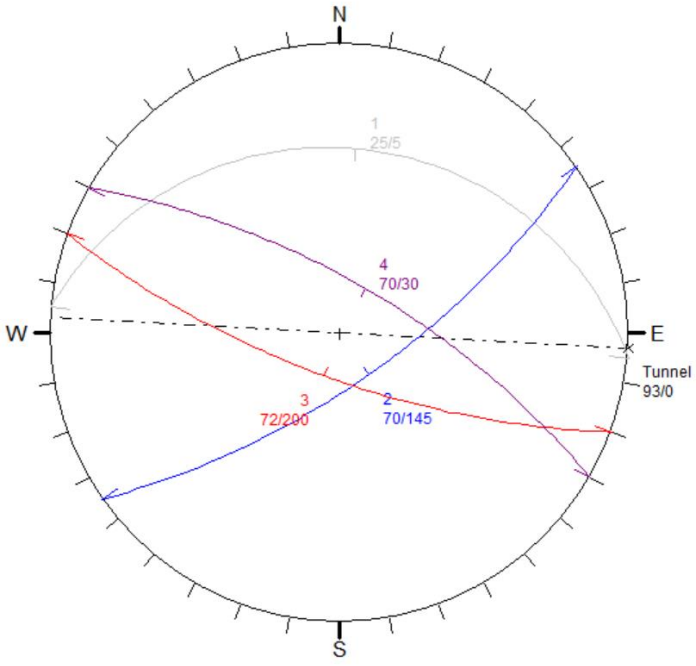


Figure 7-4 Stereographic projection of the joins J2/J3/J4

7.2 Wedge Stability Analysis

7.2.1 Tetrahedral wedges analysis

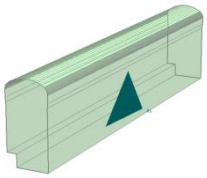
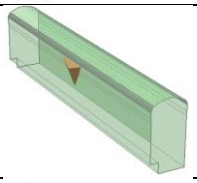
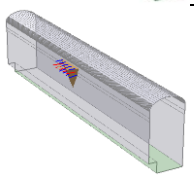
According to the wedge stability analysis, 4 groups of potential wedges may occur in the excavation periods, which are shown in Table 7-4.

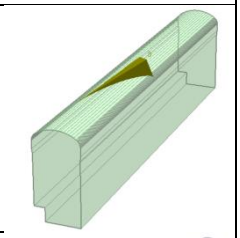
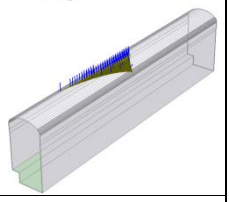
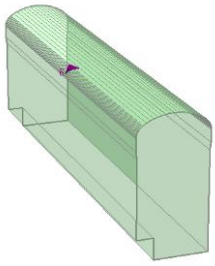
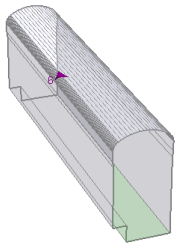
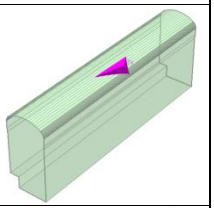
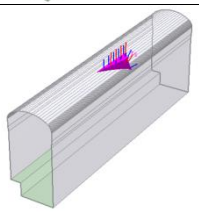
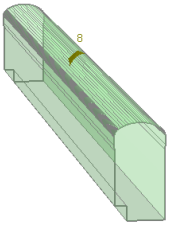
Principle for selection of potential wedge sizes: Wedge size is limited by either apex-height (= max. extent of plastic zone), or persistence of respective discontinuities, whatever produces larger wedges. Meanwhile, there should be a maximum wedge from geometry perspective.

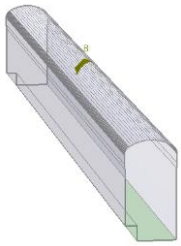
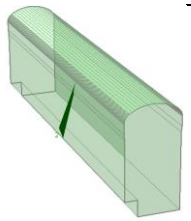
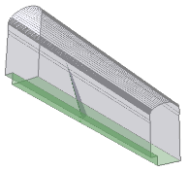
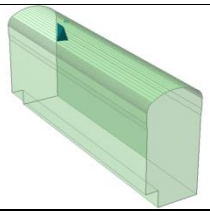
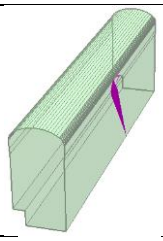
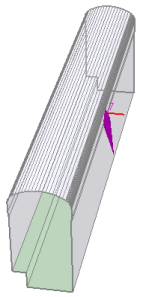
Plastic zone: according to calculation result, its depth is less than 6.0m in the roof and 6.9m in the side wall for class II surrounding rock.

Based on the above information, the potential wedges of interest are those with apex height no more than 7m for side wall, and 6m for cavern roof.

Table 7-4 Instability block search statistics of Powerhouse

No.	Fissure set number	Block volume (m^3)	Apex height (m)	Minimum persistence	Principle for selection wedge	Safety factor	Development position	Stability	Graphical presentation of block
1	J1/J2/J3	908.1	6.9	25.3	Apex height	3.05	Lower right Side wall	Stable	
2	J1/J2/J3	225.04	4.2	16.05	Apex height	1.375	Upper left side wall	Unstable	
After pattern supports						2.79		Stable	

No.	Fissure set number	Block volume (m^3)	Apex height (m)	Minimum persistence	Principle for selection wedge	Safety factor	Development position	Stability	Graphical presentation of block
3	J1/J2/J3	456.87	5.3	17.7	Geometry maximum wedge	0	Roof	Unstable	
After pattern supports						2.6		Stable	
4	J1/J2/J4	1.45	0.66	3.88	Geometry maximum wedge	0	Upper left side wall	Unstable	
After pattern supports						22.74		Stable	
5	J1/J2/J4	152.95	5.8	13.4	Apex height	1.86	Roof	Unstable	
After pattern supports						4.24		Stable	
6	J1/J3/J4	36.9	2.19	29.41	Geometry maximum wedge	0	Roof	Unstable	

No.	Fissure set number	Block volume (m^3)	Apex height (m)	Minimum persistence	Principle for selection wedge	Safety factor	Development position	Stability	Graphical presentation of block
After pattern supports						7.358		Stable	
7	J2/J3/J4	56.9	1.79	5.31	Geometry maximum wedge	0	Lower left side wall	Unstable	
After pattern supports						2.22		Stable	
8	J2/J3/J4	55.04	5.9	8.1	Apex height	1.43	Roof	Unstable	
After pattern supports						4.78		Stable	
9	J2/J3/J4	74.7	1.78	5.8	Geometry maximum wedge	0	Upper right-side wall	Unstable	
After pattern supports						2.2		Stable	

7.2.2 Prismatic wedges (Cavern roof)

As shown in the hemispherical plot below, prismatic wedges (red-shaded in plot) are developing in the cavern roof. They are formed by the Joint Sets 1,2,3 and 4, and detaching from cavern roof through the foliation jointing, which must be assumed to have nil tensile strength.

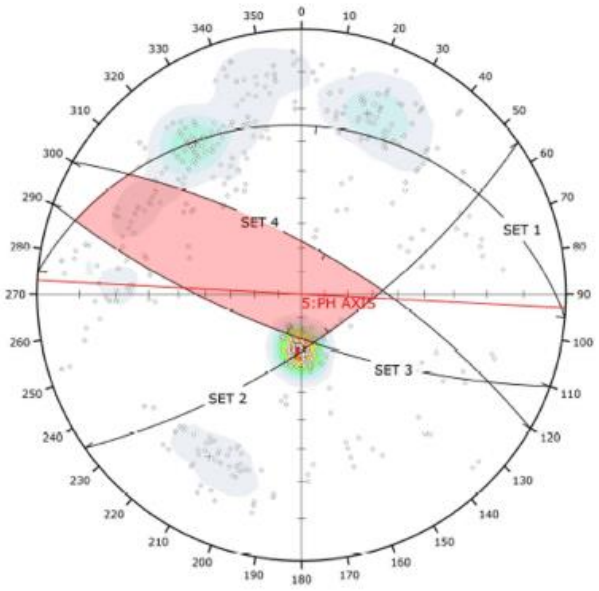


Figure 7-5 Contour and pole plot

The maximum potential prismatic wedge in the cavern roof is shown in figure 7-6. Apex height is 3.86m. Face area is 125m^2, and face perimeter is 99.8m. Volume is 316.8m^3.

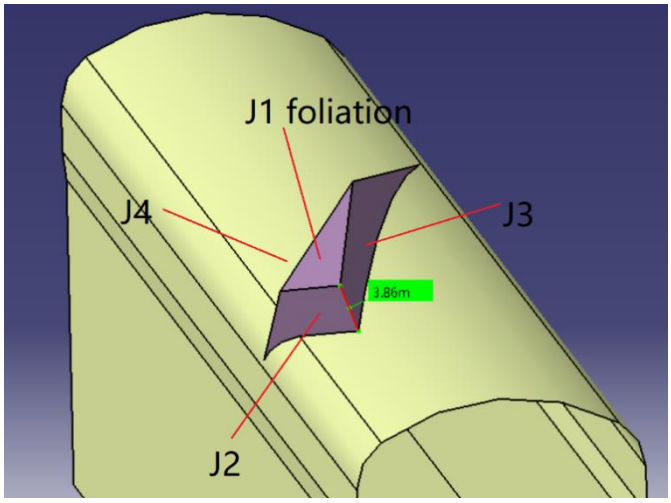


Figure 7-6 Potential prismatic wedge in the cavern roof

The weight is 8.55MN, the single rock bolt allowable bearing capacity is about 0.31MN (Dia.28mm), 0.40MN (Dia.0.40mm). Thickness and shear strength of plastic shotcrete are 0.15m and 1MPa.

According to the pattern rock bolts/dowels spacing, there are 32 pcs of rock bolts/dowels into the block. Without considering the shear strength of the discontinuities, safety factor of the block is resistance of bolt & shotcrete/ self weight of the bock=(0.31*16+0.4*16+99.8*0.15*1)/8.55=2.76. After pattern supports, the safety factor of the block can meet the requirement of the minimum safety factor of 2.0.

When apex height of wedge reaches 6.5m, the installed capacity of support is producing the required safety factor of 2.0.

7.2.3 Prismatic wedges (Cavern sidewall)

According to Annex-2 Joint Survey Report of Powerhouse Cavern. Four groups of joints with the most frequent distribution and their properties are given below:

Table 7-5 Description of joints within the powerhouse cavern surrounding rock

J1	Foliation	350-10/12-30	spacing is 0.1-0.3m, the local spacing is 0.01-0.05m, the persistence> 2m, the surface is rough and undulating
J2	joint	125-155/59-74	spacing is>20cm, the persistence is 1-5m and some >5m, the surface is rough and undulating
J3	joint	355-15/60-85	spacing is>10cm to a few meters, the persistence1-24m, the surface is rough and undulating
J4	joint	170-195/62-78	spacing is 0.2-1m, the persistence 0.5-8m, the surface is rough and undulating

And from these four groups of joints, the following four sets can be adopted for calculation.
Set 1 =355°/18° Set 2 =145°/66° Set 3 =10°/73° Set 4 =179°/70°
Mechanical parameters of joints within the powerhouse cavern surrounding rock refer to Table 7-3.

Allowable shear strength of rock dowel is taken as $\frac{1}{\sqrt{3}}$ of allowable tensile strength of rock dowel: 333.33MPa× $\frac{1}{\sqrt{3}}$ =192.45MPa

Wedge 1 is formed by J1 and J4 and joints at both ends. Wedge 2 is formed by J1 and J3 and joints at both ends. Calculation sketch of sidewall wedges 1 and 2 (prismatic wedges) are shown in Figure 7-7. Different length L are taken is consideration for unit width wedges. Calculation results are shown in table 7-6 and 7-7.

For conservative calculation, cohesion force of bottom sliding plane is only considered without cohesion force at both ends. The calculation results show that all of the prismatic wedges will be stable after pattern supports.

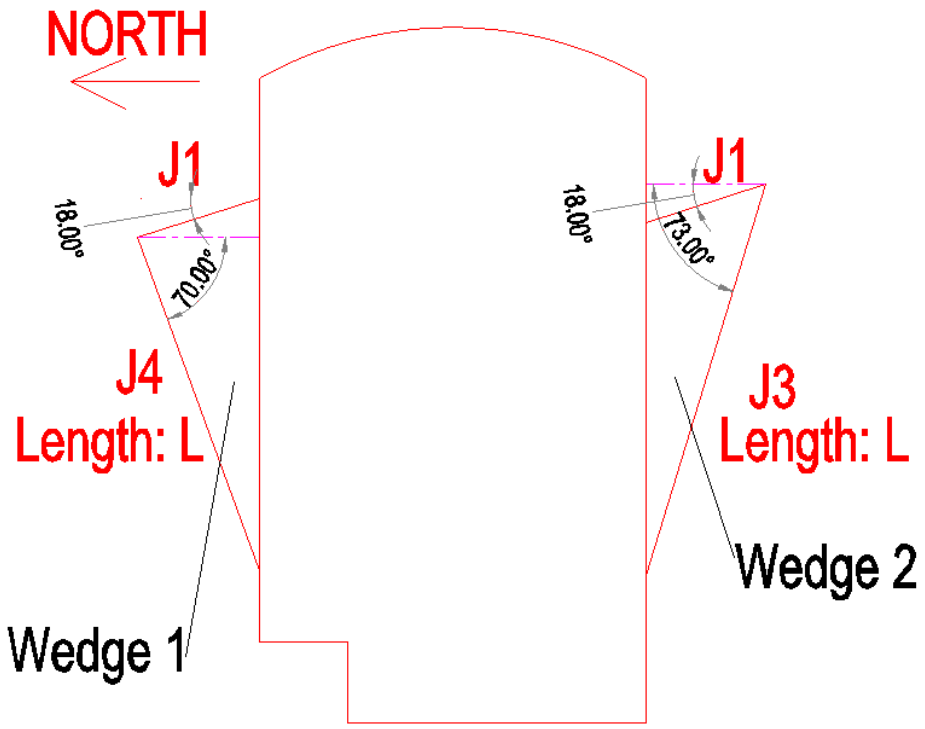


Figure 7-7 Potential prismatic wedge in the cavern sidewall

Table 7-6 wedge 1 calculation result (Unit width wedge)

Length L (m)	Area (m2)	Weight (MN)	sliding force (MN)	Friction force (MN)	Cohesion force (MN)	Quantity of rokcdowel	Anti-Shear force of rockdowel (MN)	Anti-sliding force (MN)	Safety factor
1.00	0.180	0.005	0.005	0.001	0.150	0.250	0.034	0.185	40.592
2.75	1.359	0.037	0.034	0.007	0.413	0.688	0.094	0.513	14.888
4.50	3.639	0.098	0.092	0.018	0.675	1.125	0.154	0.847	9.176
6.25	7.020	0.190	0.178	0.036	0.938	1.563	0.213	1.187	6.663
8.00	11.50	0.311	0.292	0.058	1.200	2.000	0.273	1.532	5.249

Table 7-7 wedge 2 calculation result (Unit width wedge)

Length (m)	Area (m2)	Weight (MN)	sliding force (MN)	Friction force (MN)	Cohesion force (MN)	Quantity of rockdowel	Anti-Shear force of rockdowel (MN)	Anti-sliding force (MN)	Safety factor
1.000	0.126	0.003	0.003	0.001	0.150	0.250	0.034	0.185	56.814
6.75	5.737	0.155	0.148	0.025	1.013	1.688	0.231	1.268	8.560
12.50	19.674	0.531	0.508	0.085	1.875	3.125	0.427	2.387	4.700
18.25	41.936	1.132	1.083	0.182	2.738	4.563	0.623	3.543	3.272
24.00	72.525	1.958	1.873	0.315	3.600	6.000	0.820	4.735	2.528

8 Conclusions

(1) For rock class II with basic design supports, rock bolts are quite outcropped the plastic zone, rock bolt stress at deeper section is relatively smaller than allowable stress, showing that the supports in the basic design were overdesigned and can be optimized. Contractor proposes to reduce the rock bolt length in his detail design.

(2) For rock class II with proposed supports, maximum crown and sidewall displacement is 12.5mm and 26.0mm respectively, which accounts for 0.06% and 0.1% of powerhouse span; the maximum rock dowel tensile stress is 323.6MPa, which is less than its allowable value. Therefore, the proposed supports are appropriate. Maximum crown and sidewall displacement is 13.0mm and 14.5mm respectively, which accounts for 0.08% and 0.1% of the transformer span; the maximum rock dowel

tensile stress is 315.2MPa, which is less than its allowable value. Therefore, the proposed supports are appropriate.

(3) For rock class III with proposed supports, maximum crown and sidewall displacement is 60.7mm and 135mm respectively, which accounts for 0.29% and 0.65% of powerhouse span; the maximum rock dowel tensile stress is 321.6MPa, which is less than allowable strength of rock dowel and rock bolt. Maximum crown and sidewall displacement is 44.3mm and 44.3mm respectively, which accounts for 0.28% and 0.28% of transformer span; the maximum rock dowel tensile stress is 321.6MPa, which is less than allowable strength of rock dowel and rock bolt. Therefore, the proposed supports are appropriate. The difference between the proposed scheme and basic design scheme lies in the supports at the crown, in the proposed scheme, rock bolts in vertical direction are designed to replace the rock dowels designed in the basic scheme.

(4) For wedges potentially exists in the cavern crown and sidewall, UNWEDGE analysis is carried out. The calculation results show that all of the wedges will be stable after pattern supports.

(5) During tunneling excavation of the powerhouse and transformer caverns, reasonable excavation and supporting sequence, as well as excavation height and advance, shall be followed to avoid failure of the potential unstable blocks.

9 Summary tables of determined stress and displacement data

Summary tables are provided that includes all determined stress and displacement data, the summary tables are necessary for setting up the Alert and Action Plan. Please refer to figure 9-1 and table 9-1 for displacements of multi-point extensometer, and please refer to figure 9-2 and table 9-2 for stress of two - points anchor bar stress meter and force of anchor dynamometer.

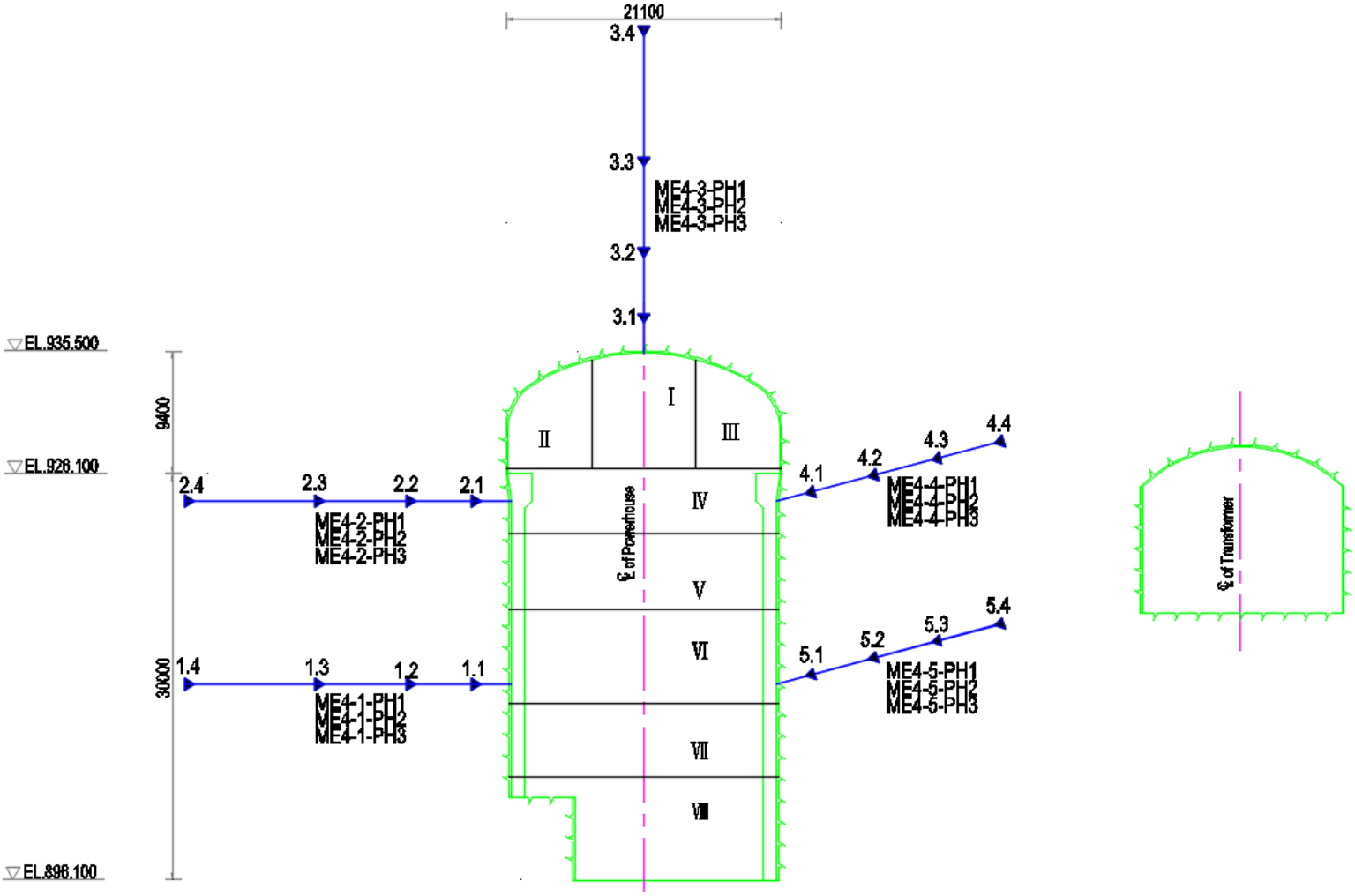


Figure 9-1 Cross section of multi-point extensometer

Table 9-1 Displacements summary of multi-point extensometer (Rock class II)

Multi-point extensometer \ Stage		Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8
ME4-1-PH1 ME4-1-PH2 ME4-1-PH3	1.1(m)						0.0119	0.0183	0.0223
	1.2(m)						0.0104	0.0159	0.0192
	1.3(m)						0.0086	0.0128	0.0153
	1.4(m)						0.0067	0.0099	0.0120
ME4-2-PH1 ME4-2-PH2 ME4-2-PH3	2.1(m)				0.0052	0.0103	0.0149	0.0184	0.0201
	2.2(m)				0.0036	0.0078	0.0118	0.0150	0.0168
	2.3(m)				0.0026	0.0057	0.0094	0.0124	0.0143
	2.4(m)				0.0018	0.0041	0.0072	0.0098	0.0114
ME4-3-PH1 ME4-3-PH2 ME4-3-PH3	3.1(m)	0.0014	0.0028	0.0053	0.0061	0.0068	0.0074	0.0078	0.0084
	3.2(m)	0.0006	0.0018	0.0038	0.0046	0.0048	0.0044	0.0046	0.0044
	3.3(m)	0.0003	0.0013	0.0028	0.0035	0.0038	0.0034	0.0036	0.0036
	3.4(m)	0.0002	0.0009	0.0021	0.0028	0.0031	0.0029	0.0032	0.0032
ME4-4-PH1 ME4-4-PH2 ME4-4-PH3	4.1(m)				0.0053	0.0088	0.0140	0.0173	0.0196
	4.2(m)				0.0037	0.0063	0.0109	0.0141	0.0164
	4.3(m)				0.0028	0.0047	0.0089	0.0119	0.0142
	4.4(m)				0.0022	0.0038	0.0074	0.0103	0.0124
ME4-5-PH1 ME4-5-PH2 ME4-5-PH3	5.1(m)						0.0115	0.0185	0.0228
	5.2(m)						0.0103	0.0162	0.0200
	5.3(m)						0.0090	0.0141	0.0177
	5.4(m)						0.0077	0.0122	0.0155

Table 9-2 Displacements summary of multi-point extensometer (Rock class III)

Multi-point extensometer \ Stage		Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8
ME4-1-PH1 ME4-1-PH2 ME4-1-PH3	1.1(m)						0.036	0.052	0.0675
	1.2(m)						0.028	0.0455	0.0525
	1.3(m)						0.024	0.0325	0.045
	1.4(m)						0.016	0.026	0.030
ME4-2-PH1 ME4-2-PH2 ME4-2-PH3	2.1(m)				0.022	0.0325	0.048	0.065	0.075
	2.2(m)				0.016	0.0225	0.036	0.052	0.060
	2.3(m)				0.012	0.0175	0.028	0.039	0.045
	2.4(m)				0.010	0.0125	0.020	0.026	0.030
ME4-3-PH1 ME4-3-PH2 ME4-3-PH3	3.1(m)	0.0075	0.0135	0.0175	0.020	0.025	0.036	0.052	0.0525
	3.2(m)	0.0030	0.0060	0.0075	0.010	0.015	0.024	0.039	0.045
	3.3(m)	0.0015	0.0030	0.0050	0.0080	0.0125	0.020	0.0325	0.0375
	3.4(m)	0.0015	0.0030	0.0025	0.0060	0.010	0.016	0.026	0.030
ME4-4-PH1 ME4-4-PH2	4.1(m)				0.020	0.0325	0.056	0.091	0.105
	4.2(m)				0.014	0.020	0.040	0.065	0.075

ME4-4-PH3	4.3(m)				0.010	0.015	0.028	0.0455	0.0525
	4.4(m)				0.080	0.0125	0.024	0.0455	0.0525
ME4-5-PH1 ME4-5-PH2 ME4-5-PH3	5.1(m)						0.040	0.065	0.0825
	5.2(m)						0.028	0.052	0.0675
	5.3(m)						0.024	0.0455	0.060
	5.4(m)						0.020	0.0325	0.045

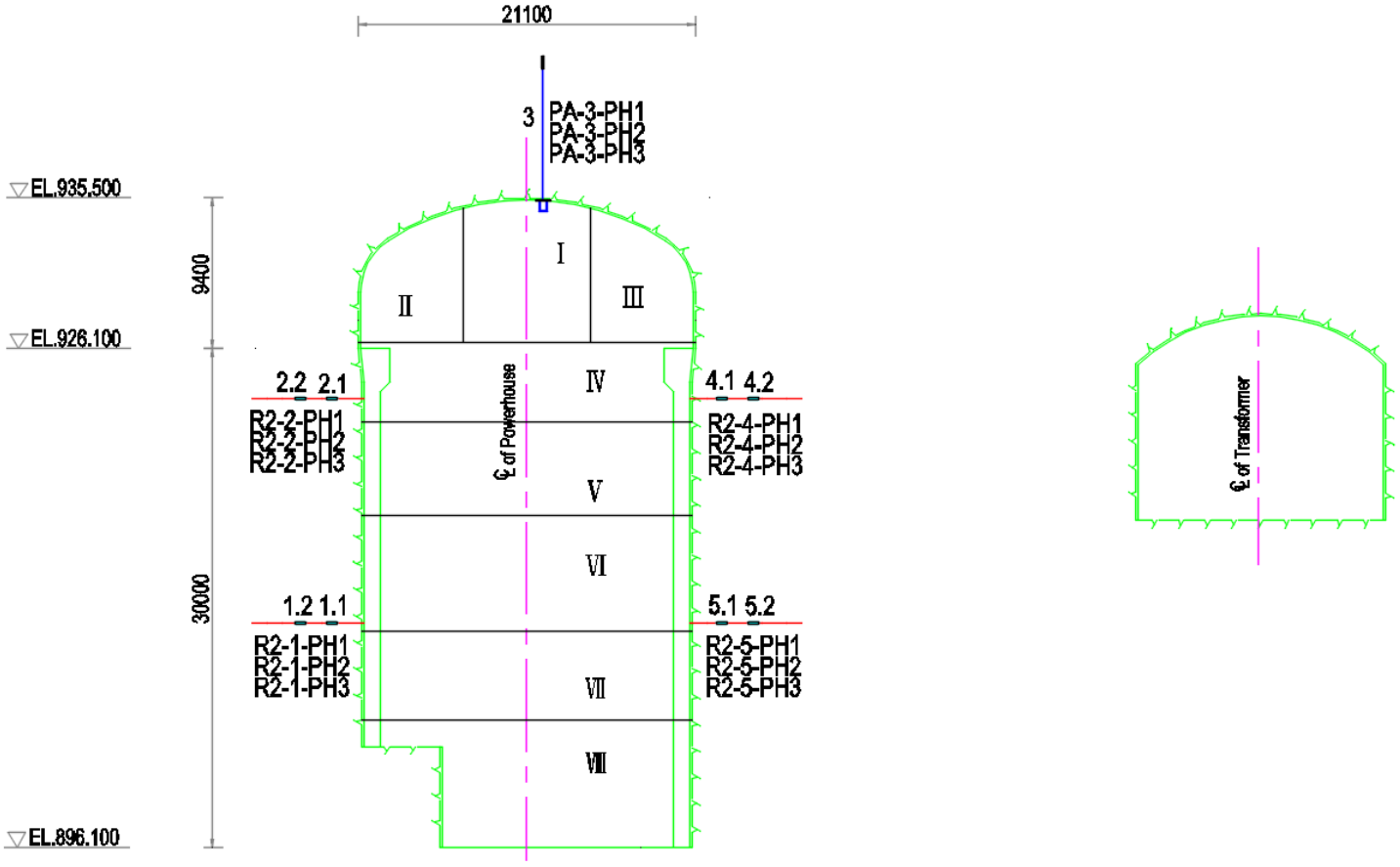


Figure 9-2 Cross section of two - points anchor bar stress meter anchor dynamometer

Table 9-3 Stress summary of two - points anchor bar stress meter and force summary of anchor dynamometer

Two-points anchor bar stress meter and Anchor dynamometer \ Stage		Stage 1	Stage 2	Stage 3	Stage 4	Stage 5	Stage 6	Stage 7	Stage 8
R2-1-PH1 R2-1-PH2 R2-1-PH3	1.1(MPa)						39.49	36.61	33.10
	1.2(MPa)						40.34	52.14	47.14
R2-2-PH1 R2-2-PH2 R2-2-PH3	2.1(MPa)				80.7	135.11	139.18	138.94	139.88
	2.2(MPa)				15.67	83.25	121.64	125.53	134.25
PA-3-PH1 PA-3-PH2 PA-3-PH3	3(MN)	0.134	0.142	0.161	0.164	0.185	0.221	0.235	0.26
R2-4-PH1 R2-4-PH2 R2-4-PH3	4.1(MPa)				45.69	100.4	105.24	120.58	131.95
	4.2(MPa)				1.96	67.64	64.24	68.44	65.33
R2-5-PH1 R2-5-PH2 R2-5-PH3	5.1(MPa)						107.6	165.71	168.11
	5.2(MPa)						41.16	42.71	37.32

10 Annex-Joints Survey Report of Powerhouse Zone